Queensboro Bridge

REPORT ON

DESIGN AND CONSTRUCTION

WILLIAM H. BURR, Consulting Engineer

BOLLER & HODGE, Consulting Engineers

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MARTIN B. BROWN
PRESS
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PARK
PLACE
NEW YORK

Wm. H. Burr,

Consulting Engineer,

Broadway and One Hundred and Seventeenth Street.

New York, N. Y., November 4, 1908.

Hon. J. W. Stevenson,

Commissioner of Bridges,

City of New York.

Dear Sir—Pursuant to action of the Board of Estimate and Apportionment, you instructed me, under date of June 9, 1908, "to examine and report upon the design and structure of the Blackwell's Island Bridge." Since that date I have been continuously prosecuting the work covered by your instructions and I respectfully submit this as my report.

I understand your instructions as to this examination to be sufficiently broad to include the general questions of safety and stability irrespective of the provisions of the specifications; in other words, that this examination is not to be limited by those provisions.

On taking up these duties I found that a part of the engineering staff of the Department of Bridges had for some time been engaged on a recomputation of the stresses of the Blackwell's Island Bridge; but in order that the duties prescribed to me might be completed as an independent and homogeneous whole, a complete analysis of the stresses in the Blackwell's Island type of bridge structure was made as set forth in Appendix I., in accordance with which an independent set of computations of stresses for the entire structure was begun and carried to completion. The results of the computations on which the conclusions of this report are based are given on sheets 1, 2, 3, 4, 5 and 6, appended hereto.

The Blackwell's Island Bridge structure is of the cantilever type, but without the usual suspended span between the extremities of the cantilever arms. The total length of the structure, exclusive of the approaches, which are not covered by this examination and report, is 3,724.5 feet, comprising 2 shore or anchor spans 469.5 feet and 459 feet long, respectively; 2 river or cantilever spans of 1,182 feet and 984 feet, respectively; and 1 island

span 630 feet in length. The width of the bridge from centre to centre of the two trusses is 60 feet. The omission of the ordinary suspended span between the cantilever arms necessitated the employment of so-called rocker arms or vertical members, having a length equal to the depth of truss and connecting the extremities of the top chords of the island cantilever arms with the extremities of the bottom chords of the other cantilever arms in the same span, thus making the trusses continuous throughout the entire distance from the Manhattan anchorage to the Queens Borough anchorage.

The contract for the superstructure of this bridge was awarded to the Pennsylvania Steel Company and executed on November 20, 1903. The entire steel work was manufactured and put in place by that company. The design as modified to accommodate four elevated railway tracks was completed in 1904.

Nickel steel was specified to be used in the eye-bars and pins of the top chords and diagonal tension members as might be ordered or approved by the Commissioner, and that metal was so used. All other parts of the structure except some small special cast iron items were of ordinary carbon structural steel.

The chemical requirements for these two grades of steel are taken as follows from the specifications:

The physical requirements of the same two grades of steel are also taken from the specifications as follows:

CHEMICAL REQUIREMENTS.

NICKEL STEEL.

	Phosphorus. P.c. Max.		Sulphur. P.c. Max.	Nickel P.c. Min.
	Basic.	Acid.		
Eye-bars and Pins	.04	.06	.05	3.25

STRUCTURAL STEEL.

Plates, Shapes, Bars and Pins	.04	.08	.05	
Rivet Steel	.04	.04	.04	
Steel Castings	.05	.08	.05	

PHYSICAL REQUIREMENTS.

SPECIMEN TESTS.

NICKEL STEEL.

	Ultimate Tensile Strength.	Elastic Limit.	Elongation.		Reduction of Area.	Character of Fracture.	Cold Bend Without Fracture.	
	Pounds per sq. in.	Pounds per sq. in.	Min. % in 8″	Min. % in 2″	%		Material less than 1 inch in thickness.	Material 1 inch or more in thickness.
Eye-bars (unannealed)........	100,000 min.	55,000 min.	$\frac{1,600,000}{\text{Ultimate Tensile.}}$	..	To be recorded.	Mostly silky, and free from coarse crystals.	180° around a pin with diameter = 3t.	
Eye-bars (annealed)...........	85,000 min.	48,000 min.	$\frac{1,600,000}{\text{Ultimate Tensile.}}$	..		do	Pieces of bar, not less than 4″ wide. 180° around pin with diameter = 2t,	
Pins (unannealed)............	90,000 min.	50,000 min.		20		do	180° around pin with diameter = 3t.	

STRUCTURAL STEEL.

Plates, Shapes and Bars for riveted work...........	60,000 desired.	30,000 min.	$\frac{1,500,000}{\text{Ultimate Tensile.}}$	..	..	Silky.	180° Flat.	180° around pin with diameter = 2t.
Eye bars and pins............	66,000 desired.	½ of ultimate.	do	22	..	do	do	do
Rivet Steel..................	50,000 desired.	½ of ultimate.	do	..	..	do	180° Flat.	
Rivet Steel, nicked and bent around bar of same diameter as rivet rod...	....	....	....	..	..	Fine, silky, gradual break—uniform fracture.		
Steel Castings (annealed)	65,000 min.	½ of ultimate.	....	18	..	Silky or fine granular.	90° around pin with diameter = 3t.	

t is the thickness of the specimen.

PHYSICAL REQUIREMENTS.

FULL SIZE TESTS.

NICKEL STEEL.

	Ultimate Tensile Strength.	Elastic Limit.	Elongation.	Reduction of Area.	Character of Fracture.	Cold Bend Without Fracture.
	Pounds per sq. inch	Pounds per sq. inch		%		
Eye-bars	....	....	....	To be recorded.	....	180° around pin with diameter = 3t.
Eye-bars (annealed)	85,000 min.	48,000 min.	9% in 18 feet, including fracture.		Mostly silky and free from coarse crystals	In the neck of the bar, 90° around pin with diameter = $2\frac{1}{2}$t.

STRUCTURAL STEEL.

Eye-bars	....	....	....	......	....	180° around pin with diameter = 2t.
Eye-bars (annealed)	56,000 min.	½ of Ultimate.	10% in body of bar.	......	Fine, silky.	
Angles ¾″ or less in thickness	....	....	....	......	....	Open flat, under blows of a hammer.
Angles ½″ or less in thickness	....	....	....	......	....	Bend shut, under blows of a hammer.

t is the thickness of the piece.

The original design of this structure contemplated but two elevated railway tracks, as the original specifications called for the following "congested" live load of 12,600 lbs. per linear foot of bridge:

2 Elevated Railway Tracks, at 1,700 lbs. p. l. f.	3,400 lbs.
4 Trolley Railway Tracks, at 1,000 lbs. p. l. f.	4,000 lbs.
35.5 ft. roadway, at 100 lbs. per sq. ft.	3,550 lbs.
2-11 ft. sidewalks, at 75 lbs. per sq. ft.	1,650 lbs.
	12,600 lbs.

A "regular" live load of one-half of the above, i. e., 6,300 lbs. per lin. ft. of bridge was further specified. In consequence of adding two more lines of elevated railway to the capacity of the bridge in September 1904, the preceding "congested" and "regular" loads were increased to 16,000 lbs. and 8,000 lbs. respectively.

The manner of using these two classes of loads, as well as the wind loading and certain other load concentrations, for determining the stresses in all parts of the structure is prescribed in the specifications as follows:

For the main members of the trusses and the towers:

(A) A load of 8,000 pounds per lin. ft, of bridge as "regular," or

(B) 16,000 pounds per lin. ft. of bridge as "congested" traffic.

For the secondary members of the trusses, the floor beams and the floor system:

(C) On each of the four elevated railroad tracks a load of 52 tons on 4 axles, 6 ft. plus 10 ft. plus 6 ft. apart (the motor ends of 2 motor cars of the Interborough Rapid Transit Co.)

(D) On each street car track either a load of 26 tons on 2 axles 10 ft. apart or a load of 1,800 pounds per lin. ft. of track.

(E) On any part of the roadway a load of 24 tons on 2 axles 10 ft. apart and 5 ft. gauge (assumed to occupy a width of 12 ft. and a length of 30 ft.), and upon the remaining portion of the floor a load of 100 pounds per square foot.

(F) On the footwalks a load of 100 pounds per square foot.

(38) The wind pressure shall be assumed as a static load of 1,000 lbs. per lin. foot and an additional moving load acting in either direction horizontally with 1,000 pounds per lin. ft.

For the proper proportioning of the parts of the structure, the following greatest permissible unit stresses in lbs. per sq. in. were provided for the various parts of the structure indicated.

	For Dead Load and Regular Live Load or for Dead Load and Wind.	For Dead Load and Congested Live Load.
	Pounds per Square Inch.	
a) For Nickel Steel in Eye Bars and Pins—		
Tension	30,000	39,000
Shear on pins	20,000	24,000
Bearing on diameter of pins	40,000	48,000
Bending on outer fiber of pins	40,000	48,000
(*b*) For Structural Steel in Main Members of Trusses, Towers and Bracing—		
Tension	20,000	24,000
Compression	$20{,}000-90\frac{l^*}{r}$	$24{,}000-100\frac{l^*}{r}$
Shear on shop rivets, bolts and pins	13,000	16,000
Bearing on diameter of shop rivets, bolts and pins	25,000	30,000
Bending on outer fiber of pins	25,000	30,000

	Pounds per Square Inch.
(*c*) For Structural Steel in Secondary Members of Trusses—	
Tension in subverticals (hangers)	18,000
Compression in subdiagonals	$18{,}000-80\frac{l^*}{r}$
Shear on shop rivets and bolts	12,000
Bearing on diameter of shop rivets and bolts	24,000
(*d*) For Structural Steel in Floor System of Roadway and Footways and in all Floor Beams—	
Tension chords	15,000
Shear on shop rivets, bolts and web-plate net section	10,000
Bearing on shop rivets and bolts	20,000
(*e*) For Structural Steel in Floor System (including brackets) for Railroad and Trolley Tracks—	
Tension chords	10,000
Shear on shop rivets, bolts and web-plate net section	7,000
Bearing on shop rivets and bolts	14,000

* Where l=length and r=least radius of gyration both in inches.

A reduction in unit stresses must be made for all members not vertical, to allow for outer fiber stresses caused by the bending of the member under its own weight.

Further provisions for the proportioning of parts required in this examination are:

"Members subject to reversals of strains shall be proportioned for each kind of strain, and the section shall be determined by the strains requiring the greater net area.

"For combined strains due to dead load, 'regular' live load and wind, the unit strains given above may be increased 20 per cent.

"No compression member shall have a length exceeding 100 times its least radius of gyration, excepting those for bracing, which may have a length not exceeding 120 times the least radius of gyration."

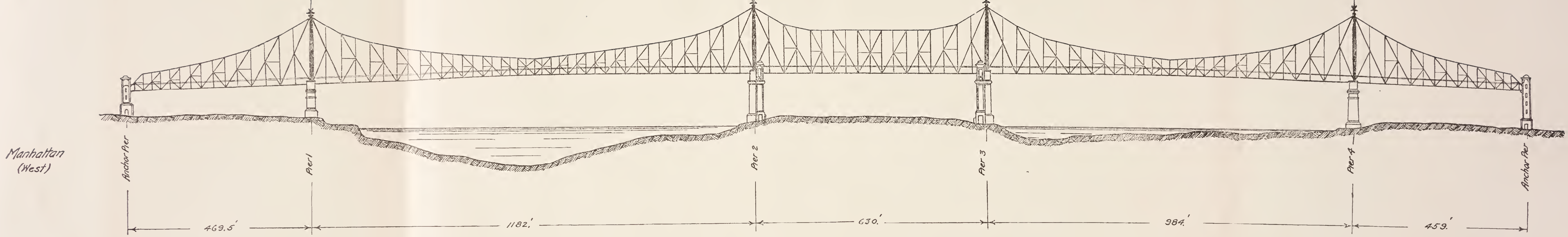
The remaining provisions of the specifications need not be quoted for the purposes of this report, but they are reasonably full and comprehensive and well calculated to accomplish the ends desired.

The transverse bracing of this bridge is designed to transfer all wind load against the upper part of the structure to the lower chord. All wind truss stresses will therefore be found in the plane or surface of the latter.

These specifications were prepared for the design of a bridge structure of unusual magnitude, the greatest length of span being 1,200 ft., and the volume of traffic to be accommodated is of extraordinary proportions. Fig. 1 shows a skeleton elevation of the entire structure between the Manhattan and Queens Borough anchorages. Fig. 2, on the other hand, shows a typical cross-section of the bridge as contempated under the specifications. The bridge has two decks or traffic carrying platforms, one along the lower chord accommodating four trolley railway tracks, two outside of the trusses and two inside adjacent to those trusses, with a 35.5 ft. roadway between the latter. The upper deck was intended under the specifications to accommodate four lines of elevated railway track between the trusses and two sidewalks, each 11 ft. wide, supported on cantilever brackets outside of the trusses.

As there are no elevated railway connections at the Queens Borough end of the bridge and no immediate prospect of such connections, the bridge will not be required to carry any elevated railway traffic for some indefinite period after its completion. Furthermore, it is still more uncertain when the growth of traffic which this bridge must accommodate will require the use of four railway tracks on the upper deck. It was decided therefore to place in position the stringers only of those tracks for the

present and to use the pair adjacent to each truss to support the sidewalk, and thus save the overhanging cantilever construction of the latter outside of the trusses, until the bridge should be required to carry full elevated railway traffic. The condition under which the bridge would be completed, therefore, is shown by the typical cross-section, Fig. 3.



Manhattan
(West)
Anchor Pier
Pier 1
Pier 2
Pier 3
Pier 4
Anchor Pier
469.5'
1182.'
630.'
984.'
459.'

Fig. 1.

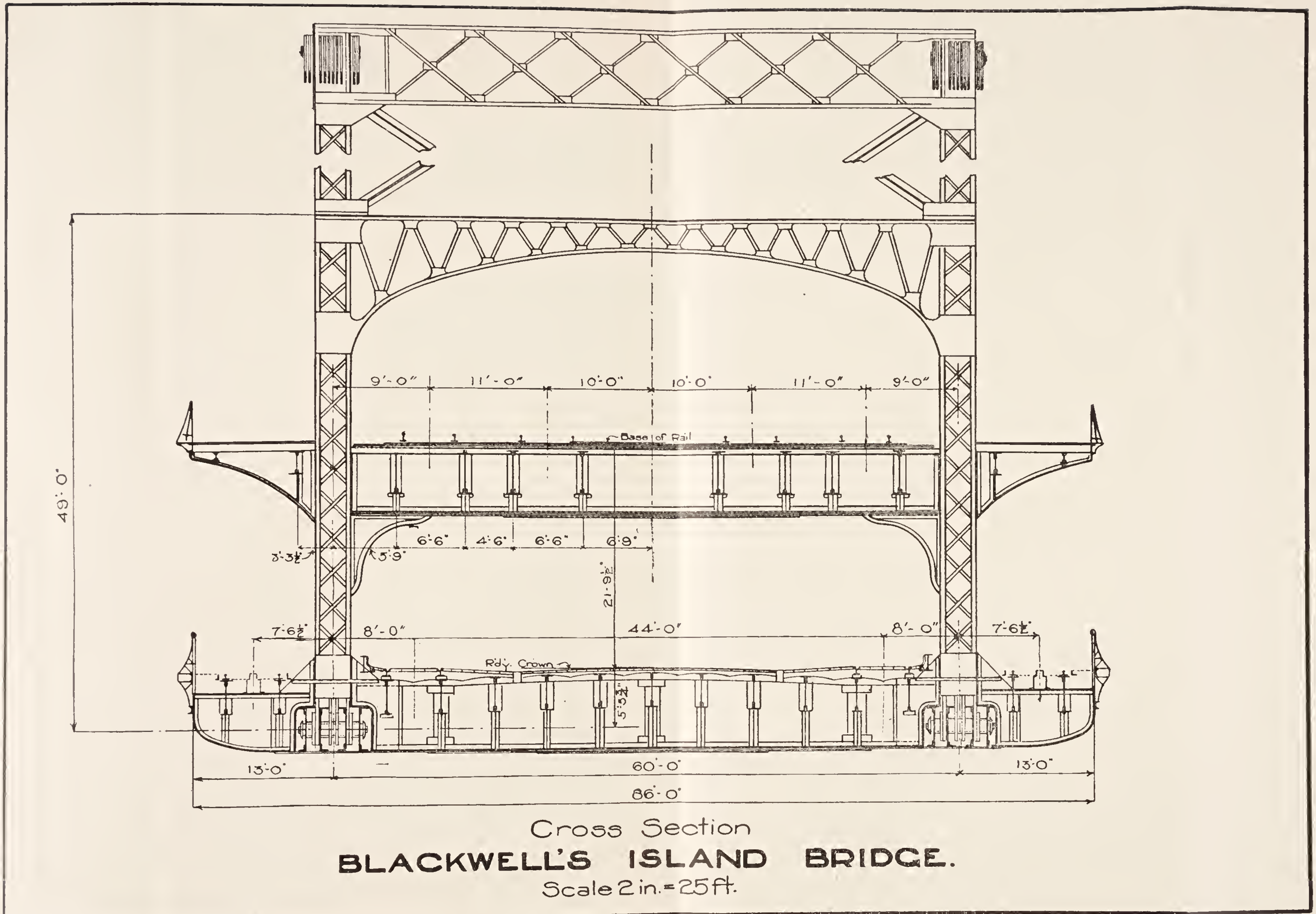

Cross Section
BLACKWELL'S ISLAND BRIDGE.
Scale 2 in. = 25 ft.
9'-0"
11'-0"
10'-0"
10'-0"
11'-0"
9'-0"
Base of Rail
6'-6"
4'-6"
6'-6"
6'-9"
3'-3½"
5'-9"
21'-9½"
49'-0"
7'-6½"
8'-0"
44'-0"
8'-0"
7'-6½"
R'dy. Crown
5'-5½"
13'-0"
60'-0"
13'-0"
86'-0"

Fig. 2.

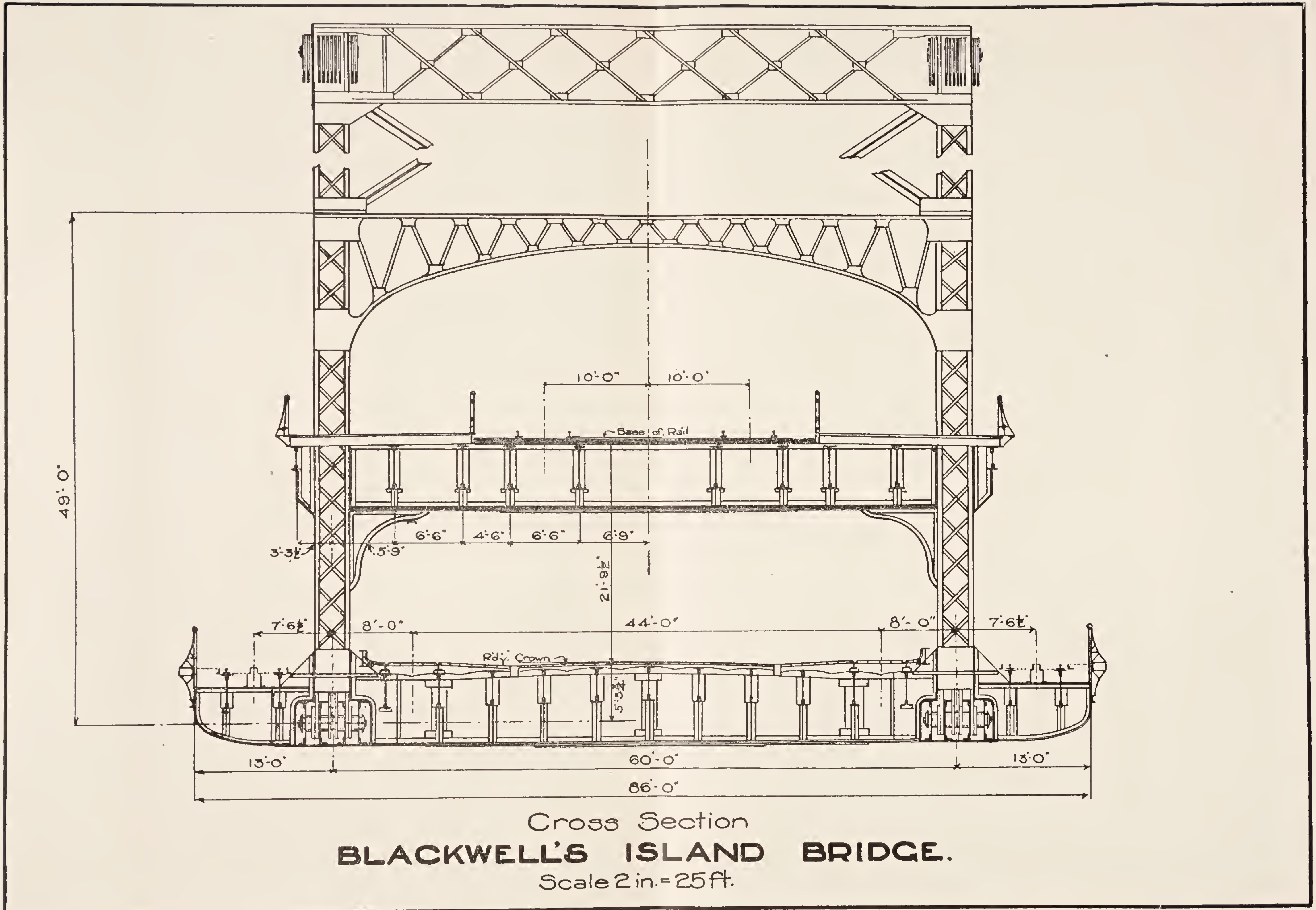

10'-0"
10'-0"
Base of Rail
6'-6"
4'-6"
6'-6"
6'-9"
3'-3½"
5'-9"
49'-0"
21'-9½"
7'-6½"
8'-0"
44'-0"
8'-0"
7'-6½"
R'd'y. Crown
5'-3½"
13'-0"
60'-0"
13'-0"
86'-0"
Cross Section
BLACKWELL'S ISLAND BRIDGE.
Scale 2 in. = 25 ft.

Fig. 3.

Proper provision for various classes of loading for a structure of such magnitude, designed to carry an extraordinary volume of traffic, with the corresponding working stresses, is largely a matter of judgment. This broad question was given much consideration by a Commission of Bridge Experts, appointed to examine and pass upon the plans for the Manhattan Bridge, under date of March 9, 1903. The Manhattan Bridge was planned for the same total traffic capacity as that of the Blackwell's Island Bridge, and that Commission recommended the same "congested" and "regular" loads of 16,000 lbs. and 8,000 lbs. per lin. ft. of bridge as prescribed for the Blackwell's Island Bridge. That Commission, consisting of Messrs. George S. Morison, C. C. Schneider, Mansfield Merriman, Henry W. Hodge and Theodore Cooper, stated, however, that the "congested" load "is a possible load which could never occur unless special pains were taken to produce it." With such an improbable but possible load, the working stresses may properly be taken substantially higher than for a "regular" load, which, while not likely to exist frequently, is far more likely to occur than the "congested." This Commission of Bridge Experts made no recommendation as to working stresses for the "congested" load, but stated "that the bridge should be so proportioned that with the 'congested' load of 16,000 lbs. per lin. ft., covering the whole bridge, combined with dead load and wind pressure, no stresses would be produced anywhere reaching the elastic limit of the material. * * * In other words, it should not be possible for such an extraordinary 'congested' load to do any permanent injury to the bridge."

Inasmuch as actual tests of full size annealed eye-bars of both ordinary structural and nickel steel have shown that the elastic limits of those members may be safely taken at about 28,000 lbs. per sq. in. and 48,000 lbs. per sq. in., respectively, it will be found that the prescribed working stresses under the congested live load in the specifications of the Blackwell's Island Bridge lie within the prescribed limits set forth by the Commission of Bridge Experts named above. In fact, it appears from the latest tests of full size carbon structural steel and nickel steel columns that the greatest prescribed working stresses for the compression members of the Blackwell's Island Bridge lie sensibly below those limits. It appears from a careful reading of the report of this Commis-

sion in connection with the specifications for the Blackwell's Island Bridge that the latter were modelled closely upon the lines laid down in the report of the Commission of Bridge Experts so far as they were applicable to the Blackwell's Island structure.

While therefore the specified loads and working stresses for the Blackwell's Island Bridge are to be considered as safe and satisfactory in the light of knowledge and precedent available when they were drawn, it is my judgment, that the experience which has accrued in connection with the actual construction of long span bridge structures within the intervening five years is such as to require some modification of the high working stresses then thought permissible. I cannot concur in the opinion of the Commission of Bridge Experts, that a possible combination of loads should be permitted to produce stresses just under the elastic limit. Whatever may have been considered permissible then, it is my judgment that the loading should be so modified now as to produce maximum stresses sensibly lower than that limit. It would be prudent to limit the greatest possible stresses produced by the "congested," dead and wind loads to three-fourths of the elastic limit.

In view of the increasing concentrated loads of nearly all kinds of traffic passing over such a structure as the Blackwell's Island Bridge, it is my judgment that the working tensile stress in subverticals or hangers could judiciously be limited to 15,000 lbs. per sq. in. instead of 18,000 lbs. per sq. in. although the use of the latter value does not prejudice the safety or durability of the structure. Such members however are subject to shock and greater concentrated loads than may be contemplated and a wide margin of safety is advisable for stresses so produced.

The provisions of the specifications both for the chemical and physical requirements of the material are in accordance with the best practice of the present day and entirely satisfactory. I have as far as possible examined the manner of inspection both in the mill and shop, and I have scrutinized carefully a mass of records of experimental data established by tests of both specimens and full size eye-bars in the course of mill inspection with satisfactory results. All of this class of evidence goes to show that the material put into the bridge was of excellent quality and fully met the requirements of the specifications.

The character of shop work is evidenced by the condition of the manufactured members in the bridge. These are largely open to ocular inspection, and I have many times been on the structure for the purpose of examining the results of the shop work. I believe it to be fully up to the requirements of the specifications and generally in accordance with the best practice of the present time. The four sub-diagonal posts C 56-L 57 and L 107-C 108 on both trusses near the main piers on Blackwell's Island were a little twisted when put in place, but this difficulty was corrected by riveting cover plates on the tops of the posts. These posts are not main truss members and the question of the safety or stability of the latter is in no way affected by them.

Many sections have been carefully calipered and the weights of many members have been computed in order to determine whether the actual dimensions of pieces as placed in the structure correspond to the requirements of the specifications, contract and shop plans. The results of these examinations have been entirely satisfactory. The actual sections of the members are generally found a little full, a margin of 2½ per cent. being allowed by specifications in accordance with common practice. The total steel in the structure as built and as determined by the actual shipping lists and scale weights confirmed by the computations mentioned above is shown by the following detailed statement:

Manhattan Anchor Span	14,501,190 lbs.
Manhattan Tower	3,705,550 lbs.
Manhattan Cantilever Arm	13,246,660 lbs.
Island Cantilever, West	12,934,040 lbs.
Island Span	21,167,080 lbs.
Island Towers	6,188,830 lbs.
Island Cantilever, East	9,370,900 lbs.
Queens Cantilever Arm	9,351,960 lbs.
Queens Anchor Arm	11,592,770 lbs.
Queens Tower	3,074,800 lbs.
Total	105,133,780 lbs.

I have made a careful examination of the entire structure as far as possible, with a view of determining whether evidence of distress of any

members exist, such as permanent distortion, loose rivets, or other evidences of overloading, misfitting or any other circumstance which might indicate over-stressing. I have not found such evidence. All parts of the structure appear to be in satisfactory condition. There are minor or small variations from alignment which are usually observed in large completed bridge work in place, but nothing whatever to indicate any inherent weakness or unsatisfactory condition.

In addition to this inspection, I have had accurate surveys made of the entire structure to determine the alignment of trusses and elevation of lower chord points in July and as late as October 26, the past month. These surveys indicate that the alignment of the trusses is entirely satisfactory, and that the deflections vertically are only those caused by the dead weight of the structure at the two dates stated, the dead weight at the later date being considerably more than at the former in consequence of the large amount of floor material put in place within the period named.

The computations for the stringers and floor beams for both upper and lower decks show that in the main, the prescribed unit stresses are not exceeded. In a large number, perhaps the majority of cases, the computed stresses are considerably below the specified limits. In a few cases of the lower floor beams and trolley stringers, the specified unit stresses are to a small extent exceeded. This excess is due to the use of less dead load than the weight of the floor as finally constructed. This excess, however, is too small to be material.

The outside trolley stringers on the lower floor of the Island span and its cantilever arms have been built in accordance with the specifications, but while they were being completed, it was thought that the heavier subway cars might at some future time run over these tracks. Consequently, the outside trolley stringers on the remainder of the bridge were cover-plated to meet the requirements of the increased loading. If, in the future, the heavier subway cars should be run over the structure, it would be necessary, as was intended, to put these cover plates on the stringers of the Island span and its cantilever arms, which may be readily done if it should ever be required.

An examination of the design and construction of the anchorages at both ends of the structure shows that weights of masonry and other portions of the anchorage and its members are sufficient and satisfactory for

their purposes. The pressures on the masonry in the main piers are also within the requirements of the specifications.

The erection stresses are now of secondary importance, if of any importance at all, as the structure is erected and the erection stresses have passed never to recur. These stresses, however, have been computed with the weight of the traveller taken at 647 tons distributed as in actual use. Two such travellers have been assumed to be in use simultaneously on the east and west cantilever arms of the Island span and in positions to produce the greatest stresses. The results of these computations show that the erection stresses in the chords could not at any stage of the construction equal those due to the completed dead weight of the structure. In a few of the diagonals in the vicinity of the main piers, the erection stresses equal those produced by the specified dead and live load. Inasmuch as these stresses exist in the structure but once and will never exist again, they have no special significance in regard to the carrying capacity of the bridge, nor has the weight of the traveller been a test of the structure comparable with the traffic loading which must be carried.

It is to be observed that the specifications do not call for any snow loading. In view of the fact that there are occasionally heavy snowfalls at New York, sometimes followed by rain and subsequent freezing, it would seem to have been advisable to make some suitable provision for this class of loading. At the same time, if the usual street cleaning removal of snow from the bridge is promptly made, it is clear that there is practically no possibility of snow load and even the regular live load concurring. The administration of this structure should, therefore, be stringently carried out to meet that requirement, otherwise an objectionable overloading is possible.

It is assumed throughout this investigation that the adjustment of the supports at the ends of the anchor arms is such that the rocker arms are subjected to no stress when the bridge is free of live load. The determination of all stresses requires that condition to be assured after the completion of the bridge.

I have had computed a complete stress sheet for the greatest possible stresses in every part or member of the entire structure using the prescribed "congested" moving load and wind loading and the actual dead weight of the structure as determined by the shipping weight of all the steel and other metal members, and by the working plans for the railways, roadways and

sidewalks of the upper and lower decks. The dead weight of the structure thus determined is composed of the items per linear foot of bridge shown in detail on the stress sheets.

The results of these computations show that there are a considerable number of the main truss members which would be overstressed under the provisions of the specifications for the assumed loading. Some of the bottom chord panels of the Island span would carry about 25 per cent. more than permitted by the specifications for compression under dead and congested live loads. The bottom chords of some other portions of the bridge would be overstressed under the same conditions up to a maximum of about 15 per cent.

Under the conditions for which this structure is designed, as already stated, the top chord and main truss members other than those of the bottom chords are not affected by the wind.

With the same conditions of loading as above, the nickel steel eye-bars in the top chords of the Island span would be overstressed about 25 per cent. as a maximum, and 20 per cent. in the top chord of the Queens cantilever arm, while the maximum overstress in the Manhattan cantilever arm and the two Island cantilever arms would be 15 per cent. to 20 per cent. The overstresses in a number of the carbon steel eye-bars would range from 10 to 15 per cent. in some members in all parts of the structure except in the Queens anchor arm where 30 per cent. is reached in one case. There would be also some similar overstresses in riveted tension members in the same parts of the bridge rising above 33 per cent. in ~~a~~ ~~single~~ two instances. Few main posts in all the bridge would be overstressed as much as 25 per cent. In the Queens anchor arm there is one post which would be overstressed 33 per cent. The Manhattan rocker arm would be overstressed nearly 20 per cent. in tension and 30 per cent. in compression, while the Queens rocker arm would be subjected to an excess of 10 per cent. in tension and 12 per cent. in compression.

Among the secondary or sub-truss members, one hanger near the centre of the Manhattan cantilever span would be overstressed about 20 per cent. and another about 10 per cent. All other hangers, sub-diagonals and sub-posts would receive practically their proper stresses only.

These greatest overstresses running with two or three exceptions not above a maximum of 25 per cent. of the stresses permitted under the specifi-

cations would not be serious if the permitted working stresses were not initially high. In fact, there are bridges in use and considered safe in which similar overstresses exist although such conditions can never be considered satisfactory. Again, as none of these excessive stresses per square inch of cross-section, even when combined with wind load stresses, exceed some of the elastic limits determined by experiment for the corresponding members in which they are found, those stresses exceed but little, if any, the limits permitted for the stresses due to the congested live, dead and wind loads of the Commission of Bridge Experts whose report has already been alluded to. In my judgment, however, as already expressed, the maximum stresses permitted in that report, under the congested, dead and wind loads, are too high, and it is further my judgment that these computations show that sufficient provision was not made in designing the cross-sections of the members of the Blackwell's Island Bridge for the two additional elevated tracks which were added to the structure in 1904.

It becomes necessary, therefore, to ascertain what is the safe maximum loading of this bridge for both the lower deck and the upper deck, including in the latter such elevated railway tracks as may be permissible. For this purpose it is considered prudent to adopt for the greatest permissible working stresses those prescribed in the specifications "For Dead Load and Regular Live Load or for Dead Load and Wind," except that for reasons given later on the working stress for columns will be taken as 20,000-50 l/r.

Substantial advantage may be gained for the structure by removing some portions of the dead load of the floors both for the upper and lower decks. An examination of the plans shows that considerable concrete under the two inside trolley tracks of the lower deck as well as some other floor details may be either omitted or modified without in any way trenching upon the capacity, solidity or strength of the lower floor. Similarly, if the two upper elevated railway tracks, including the supporting stringers added in 1904, be removed, and if the sidewalk railings of the upper deck be rearranged without affecting their capacity or strength, the dead weight of the structure may be reduced at least 1,272 pounds per lineal foot of each truss on all the cantilever arms. It is conducive to relieving the maximum stresses to permit the weight of the lower deck to remain unchanged on the Island span and the two anchor arms. The relief of dead weight on those

three parts of the bridge would then be but 380 pounds per lineal foot of truss.

In considering the maximum loading to be taken in a re-computation of stresses, it should be observed that the conditions which will produce a congested load in one direction on this or any similar great bridge will be likely to stop traffic largely or entirely in the other direction, as has frequently been observed in treatments of this question of congested loading for great bridge structures. For this reason, it is believed that 50 pounds per square foot of roadway and sidewalk is sufficient for an assumed congested load for the main truss members of the Blackwell's Island Bridge, that loading to cover any or all parts of the structure necessary to give any member its greatest possible stress.

The heaviest trolley car now running in the city with its full load weighs 62,000 pounds on four axles, and has an out to out length of 42.5 ft., thus producing an average load of 1,460 pounds per lineal foot of each track. The heaviest motor car running on either elevated or subway track when fully loaded has a weight on each of two axles of 30,800 pounds and 22,200 pounds on each of the other two. As this car has an out to out length of 51 ft., its average weight is 2,080 pounds per lineal foot of track. The heaviest trailer car now running in the city and fully loaded has a weight of 70,000 pounds, with a total length of 51.4 ft., making an average load of 1,360 pounds per lineal foot of track. An eight car train composed of five motor cars and three trailers giving an average weight of 1,810 pounds per lineal foot of track will be taken for the moving load on two elevated railway tracks of the upper deck.

All dense trolley or elevated railway traffic on the East River bridges of this city must be effectively controlled by proper signals and continuous inspection, as experience has demonstrated on the Brooklyn and Williamsburg bridges. In computing the stresses in the main truss members of the Blackwell's Island Bridge the same control of the trolley traffic will be assumed as that exercised on the Brooklyn Bridge and on the Williamsburg Bridge, two succeeding trolley cars being separated by a clear space of about two car lengths. Similarly, the control of the traffic on the two elevated tracks would be secured by making the minimum distance between the heads of trains 1,000 ft. as is now done on the Brooklyn Bridge. The maximum stresses produced in the trusses

of the Blackwell's Island Bridge however are found by placing one 8-car train with an average of 1,810 lbs. p. l. f. of track at the center of each of the Manhattan and Queens cantilever spans, making the heads of trains about 1,722 ft. apart. Any other arrangement of trains with a distance from center to center not less than 1,000 ft. will produce less stresses in the truss members than that just described.

This total moving load representing the maximum permissible traffic for the Blackwell's Island Bridge, arranged with two elevated railway tracks and constituting what may be called a congested live load will be as follows:

2 Elevated 8-car trains, at 1,810 lbs.	3,620 lbs. p. l. f.
4 Trolley tracks, 1/3 by 4 by 1,460 lbs.	1,947 lbs. p. l. f.
35.5 ft. Roadway, at 50 lbs. per sq. ft.	1,775 lbs. p. l. f.
22 ft. Sidewalk, at 50 lbs. per sq. ft.	1,100 lbs. p. l. f.
	8,442 lbs. p. l. f.

This total load of 8,442 lbs. is for each linear foot of bridge covered simultaneously by the four classes of traffic, each truss carrying one-half of this total.

The stresses computed for all the main truss members is shown on sheet No. 3. The positions of trains required to produce these greatest stresses are shown on sheets Nos. 4 and 5.

Comparing results with the specified permissible unit stresses given on page 52 of the specifications and reproduced in the earlier part of this report for dead load and regular live load, excepting those for columns already alluded to, this stress sheet shows that no unit compressive stress in all the trusses exceeds the prescribed working limit. Nor is there but one instance where nickel steel eye-bars show stresses as much as 4 per cent., or, but three instances as much as 3½ per cent. above prescribed value. Similarly, but one carbon steel tension member shows 4½ per cent. overstress. These overstresses are too small to be of moment and they may be ignored.

It has been shown by actual computations, therefore, that the Blackwell's Island Bridge has a safe and satisfactory capacity for carrying a volume of traffic under the conditions of control found advisable by

experience on the Brooklyn and Williamsburgh Bridges, sufficient for a considerable future period and perhaps for a long future period. No elevated traffic can pass the structure for some time to come as there are no connections for such traffic at the Queens end of the bridge. Whenever such traffic may be required, the two elevated tracks on the upper deck will serve that purpose. The question of further accommodation for lines of traffic on the upper deck may judiciously be left for later consideration when demanded by future requirements and when the character of traffic to be accommodated at that time shall become known. It is only necessary so to adjust the accommodations for such future developments as will keep the unit stresses in the main truss members within the limits prescribed in the specifications for regular live and dead loads or for those loads combined with the wind loading.

The formula given in the specifications for the safe carrying capacity of columns or posts of carbon structural steel members is 20,000-90 l/r lbs. per sq. in. Recent tests of full size members of this class indicate clearly that this formula gives results lower than need be taken for this class of members, and hence, in computing the proper working stresses for the structural steel members of this bridge, the value 20,000-50 l/r has been used. The ultimate carrying capacity, which is practically the elastic limit, of such columns with a ratio of length over radius of gyration of 40 to 50 is not less than 31,000 to 32,000 lbs. per sq. in., nor less than 28,000 to 29,000 lbs. per sq. in. for a ratio of length over radius of gyration of 90.

I recommend that the Department of Bridges have made and tested to destruction not less than six pairs of structural steel compression members, each pair being duplicates, similar in cross-section to the typical compression members in the chords and web members of this bridge, the areas of cross-section of such columns to be the greatest capable of being tested in the testing machines of the largest capacity in this country, such test columns to have pin heads. Such tests could be made within a comparatively short time; and if the test columns are judiciously designed with a range of length over radius of gyration varying from 40 to 100, valuable and accurate information as to the safe carrying capacity of such members would be disclosed. It is not necessary, however desirable it might be, to test columns of the greatest

area of cross-section in the trusses, which are far beyond the maximum capacity of any testing machine now available.

Approximate computations only have been made for the secondary stresses due to the distortion of the trusses by the live load, and by the variation of temperature. Although such computations may readily be made, it is a matter of grave doubt whether these stresses ever exist to the extent indicated by computations. When secondary stresses are computed at truss joints, it is essentially impossible to make due allowance for the presence of the heavy joint plates and other details which greatly reduce the stresses computed for the sections of the members themselves. As a matter of fact, observations on wrecked trusses show that in well designed work the joints at which the secondary stresses are supposed to be greatest are the parts frequently showing the least distress and the greatest rigidity or the least departure from their normal condition. The approximate computations made for these secondary stresses in a considerable number of truss members show that the computed intensities are below 3,000 to 3,500 lbs. per sq. in. As these maximum intensities exist in extreme fibres only and probably not to the extent shown by the computations, it is clear that they can have no material effect upon the safety or stability of the truss members.

The stresses arising from the bending of horizontal or inclined truss members due to their own weight are of a much more definite character. Computations of these own weight bending stresses have been made on a large number of truss members and they may in some cases run as high as 3,000 lbs. per sq. in. to possibly 4,000 lbs. per sq. in., although generally of lower value than the former. Again as these are extreme fibre stresses only and for short parts of the members in question, they may be disregarded as having no material effect upon the proper safe or working value of the members in question.

There are also some stresses which may be called secondary which are due to the acceleration and retardation of moving trains upon the elevated railway tracks which are supported midway of the vertical web members, but they are of such uncertain values, scarcely subject to even approximate computation, that they may be legitimately considered as incidental stresses covered by the margin allowed in determining proper working stresses.

The wind stresses shown on the stress sheet, Sheet No. 3, are compiled from the prescribed wind loads and in the manner prescribed by the specifications. It will be observed that chord stresses only are given as the design of the bridge trusses with their transverse bracing is such that all wind loads are carried down to the lower deck over which a rigid buckle plate floor is found. This buckle plate floor and the lower chords constitute to a material extent a great plate girder to resist the horizontal wind forces. No web stresses are therefore to be determined. As a matter of fact it is a question whether even the full computed chord wind stresses can exist.

Adverse comment has been made on the heavy compression lower chords of this bridge and their design therefore has been scrutinized with great care. Each chord is composed of two parallel and independent, built-up channel compression members connected together with top and bottom 15-inch by ½-inch batten or tie plates. Each member of this pair of compression chords is composed of two built-up channels with vertical web plates 48 inches deep and of varying thicknesses to afford the proper area of section, each web having 8-inch by 6-inch angles top and bottom. Each of these pairs of built-up channels is latticed, top and bottom, with a double system of 5-inch by ⅝-inch lacing bars held by two rivets at each end and one at each of the intersections. The entire chord composed of each of these two units has therefore a total width of about 82 inches.

The radius of gyration of each of these chord units about its horizontal neutral axis is about 14 inches, while the radius of the same section about its vertical neutral axis is about 12 inches. The numerous top and bottom batten or tie plates certainly give material horizontal stiffness to the two units although not as much as would heavy lattice bracing. It is reasonable and safe to give the horizontal radius of gyration of this double column section a value of 14 inches identical with that of its vertical radius, and this has been done in the computations. It will be found by examination of the stress sheet that there are other compression members of the trusses which sustain higher unit stresses with a greater ratio of length over radius of gyration. There can be no apprehension, therefore, that these lower chord compression members are in any way unsafe or of less unit carrying capacity than other main compression members of the trusses.

No criticism of these lower chord compression members would probably have been made except for the failure of chord sections of somewhat similar

general shape of section in the Quebec Bridge. The similarity, however, lies only in the general form of section of the component parts. The spacing details of the Blackwell's Island chord sections, consisting of heavy lattice bars, batten and tie plates, and transverse diaphragms, are relatively far heavier, stiffer and stronger than corresponding details in the Quebec trusses; indeed, in the latter, there were no transverse diaphragms such as are found in the Blackwell's Island Bridge. There is therefore little or no similarity as to the unit carrying capacity of the compression chord members in the two bridges.

The deflections of the extremities of the cantilever arms under the moving load shown on the stress sheet accompanying this report may be as great as 23 inches. In computing this deflection the gross section of riveted tension members has been used, and a modulus of elasticity for both carbon and nickel steel of 28,000,000 pounds per square inch. This deflection would be less if the depth of trusses were greater. Substantial economy would have been attained if the trusses had been designed with a greater depth and with longer panels in the main truss system, at the same time gaining greater stiffness.

The lateral stiffness of a cantilever structure, especially of this magnitude, is of much importance and it is affected greatly by the horizontal width between trusses which in the present instance is 60 ft. A comparison of the length of the Manhattan cantilever arm, 591 ft., with this 60 ft. width between trusses shows that the latter dimension has been judiciously chosen, giving to the structure a proper lateral stiffness, especially in connection with the buckle plate floor.

Conclusions.

First—The specifications for the chemical and physical requirements of both the nickel and carbon steels employed in the structure are satisfactory and in accord with the best practice of the present time.

Second—Both the shop and mill inspection were efficiently performed, resulting in securing excellent quality of material and the fabrication of truss members of good quality and accurate dimensions.

Third—The various members of the structure possess the full sections required by the unit stresses and the working plans, and the shipping

weights correspond correctly to those sections as well as to the computed weights.

Fourth—The erection was successfully and satisfactorily performed, leaving the trusses in correct alignment and elevation.

Fifth—Computations in accordance with the specifications for the maximum floor loads show that the capacities of the floors for both the upper and lower decks are satisfactory.

Sixth—Computations for all the main truss members of the bridge show that the stresses produced by the prescribed congested live load, combined with the dead load or with the dead load and wind loading are higher than prescribed as permissible in the specifications, and higher than prudent to permit, although practically not in excess of the limits approved by the Commission of Expert Engineers in 1903.

Seventh—The stresses disclosed by the stress sheet submitted with this report show that a controlled traffic on the four trolley lines of the lower deck and on two elevated railways of the upper deck carrying the heaviest cars of their classes now in use in the City of New York together with a vehicular traffic on the roadway and two loaded sidewalks may be permitted without exceeding the specified unit stresses for the regular live load and dead load and without exceeding the safe limits of stresses for such a structure, provided the re-arrangement of the floor of the lower deck and the removal of the two elevated railway tracks on the upper deck together with the re-arrangement of the sidewalk details be made as indicated in this report so as to reduce the dead load by at least 1,272 lbs. p. l. f. of each truss of the cantilever arms and 380 lbs. p. l. f. of each truss of the Island span and anchor arms. This re-arrangement and reduction of dead load can now be made without material delay in the opening of the bridge for traffic. The capacity so afforded is satisfactory and sufficient for a considerable future period. Any further use of the upper deck for elevated railway or other purposes should be deferred until the development of traffic in the future may make it necessary, and until it shall be determined what character of traffic must then be accommodated, but that adjustment to the future traffic should not be such as to produce greater unit stresses than those approved in this report.

Eighth—The distance between trusses is suitable to this type of structure and such as to secure satisfactory lateral stability, especially in connection with the buckle plate floor.

Respectfully submitted,

WM. H. BURR,
Cons. Engr.

APPENDIX I.

The Cantilever Without Suspended Span.

A cantilever structure comprising two anchor arms, two cantilever spans and an anchor span between the two latter, like the Blackwell's Island Bridge, becomes continuous from anchorage to anchorage when the suspended spans are displaced by rocker arms, vertical in the present case, connecting the upper and lower extremities of two adjacent cantilever arms. The stresses in the different members of the trusses cannot, therefore, be determined by the ordinary procedures applicable to statically determinate structures. It is necessary first to find the stresses (reactions or shears), in the rocker arms by the aid of formulæ involving the elastic properties of the materials of the structure, the latter being loaded in any manner whatever. The continuity of the trusses makes it necessary to recognize, in the analysis, the simultaneous existence of the two rocker arm reactions or shears.

The complete determination of the general values of those reactions or shears is as follows.

The Two Reactions or Shears R^1_2 and R^1_4.

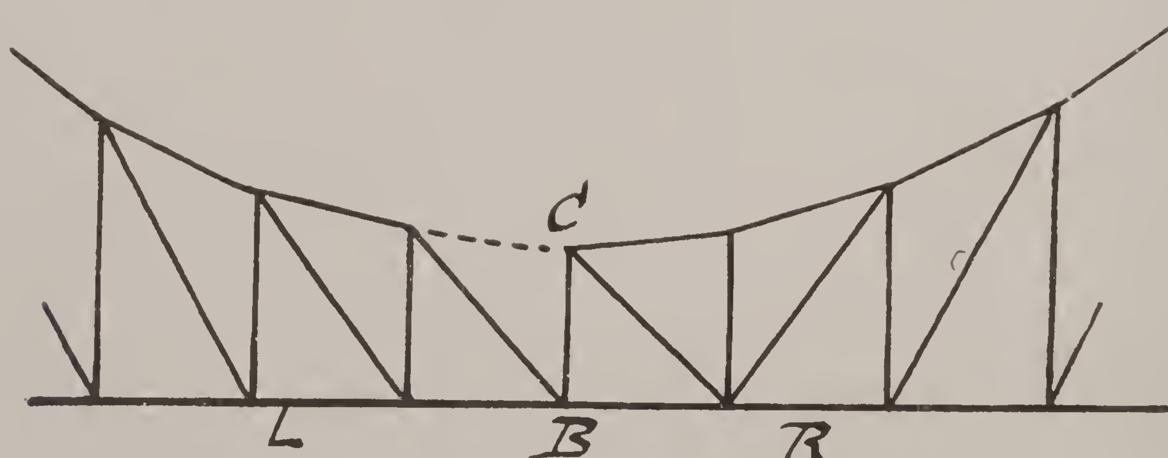


Fig. 1.

To make the case general it will be assumed that the lengths of cantilever arms on the right and left, respectively, are unequal and represented by l_R and l_L. The length of the rocker arm CB is h, and its area of cross section A. The line LR in Fig. 2 is supposed to represent the lines of

the cantilever arms, at the same elevation, before the structure is so loaded as to cause unequal deflections.

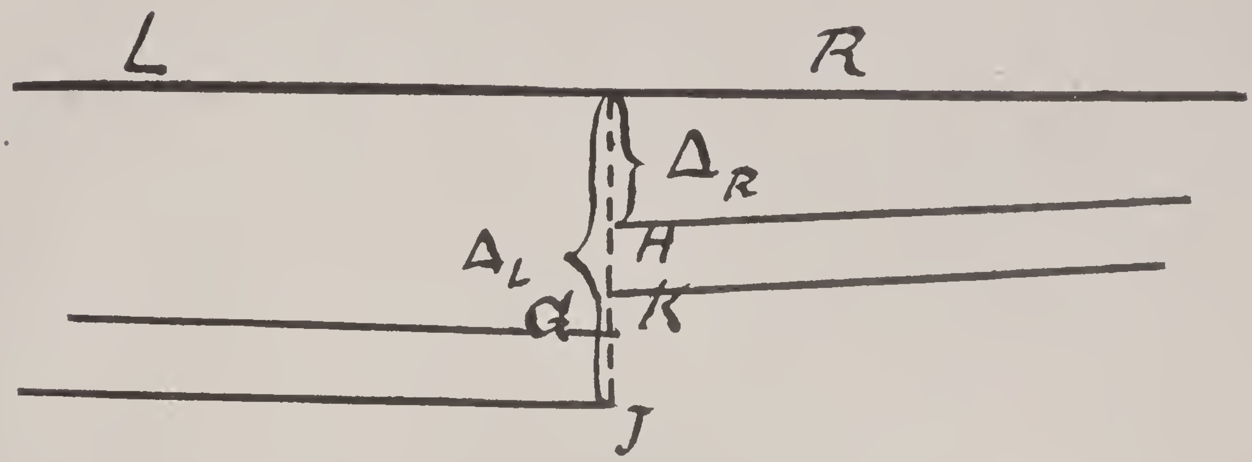


Fig. 2.

The principal quantity to be found is the force or reaction exerted between the ends of the cantilevers through the rocker arm BC and when loaded in any arbitrary manner. All other desired quantities can then be at once determined.

If all connection between the extremities of the cantilever arms is cut when those arms are loaded in any arbitrary manner, in general the end of the right arm will have the deflection Δ_R while the deflection of the left end will be Δ_L. If, on the other hand, the ends are connected by an elastic member they will deflect to some points G and K between the lower limits of Δ_R and Δ_L. Let the force or reaction exerted in this elastic connection be represented by R_2^I. Also in Fig. 2 let Δ_R^I represent HK while Δ_L^I represents JG, remembering that HK and JG are the downward and upward deflections produced by the force or reaction R_2^I acting at the end of each cantilever arm as if the latter were free of any end connection. Then let ω be any arbitrary load supposed to act at the extremity of each free cantilever arm and to produce there the deflections δ_R and δ_L in the right and left arms respectively. Since these deflections are proportional to the forces producing them:

$$\frac{R_2^I}{\omega}=\frac{\Delta_R^I}{\delta_R} \text{ and } \frac{R_2^I}{\omega}=\frac{\Delta_L^I}{\delta_L} \quad \ldots\ldots\ldots\ldots \text{I}$$

The change of length of the rocker arm CB caused by the stress in it is: $\frac{R_2^I h}{EA}$; E being the coefficient of elasticity.

In this analysis downward deflection will be considered positive and upward deflection negative. In the case of Fig. 2 Δ_L^I would be negative.

Furthermore: $\Delta_R^I - \Delta_L^I = \Delta_L - \Delta_R - \frac{R_2^I h}{EA}$ 2

Substituting for Δ_R^I and Δ_L^I from Eq. 1 in Eq. 2;

$$R_2^I = \frac{(\Delta_L - \Delta_R)\,\omega}{\delta_R - \delta_L}\left[\frac{1}{1 + \frac{h\,\omega}{EA\,(\delta_R - \delta_L)}}\right] \qquad 3$$

If Fig. 2 be supposed applied to span l_4, Δ_R will be Δ_L of Eq. 3, while Δ_L will be Δ_R of that equation in consequence of the symmetry of arrangement of spans l_2, l_3 and l_4.

If the division indicated in the bracket of the second member of Eq. 3, be carried out for two terms only, there may be written approximately:

$$R_2^I = \frac{(\Delta_L - \Delta_R)\,\omega}{\delta_R - \delta_L}\left[1 - \frac{h\,\omega}{EA\,(\delta_R - \delta_L)}\right] \qquad 4$$

There is no special advantage in the form of Eq. 4. Equation 3 was used in all the computations covered by this report.

The fraction whose numerator is $h\,\omega$ in the bracket of the second members of Eqs. 3 and 4 is the allowance or correction to be made in the value of the reaction R_2^I for the extension or compression of the rocker arm, and it may be omitted if that member is very short and of large cross sectional area.

SPECIAL VALUES OF REACTION R_2^I.

For the purposes of this analysis an upward reaction will be considered negative and a downward reaction positive.

If the load producing R_2^I is wholly on the left cantilever arm, so that $\Delta_R = 0$, Δ_L only existing, Eq. 3 at once gives;

$$R_2^I = \frac{\Delta_L\,\omega}{\delta_R - \delta_L}\left[\frac{1}{1 + \frac{h\,\omega}{EA\,(\delta_R - \delta_L)}}\right] \qquad 5$$

On the other hand, when the right arm is loaded so that $\Delta_L = 0$, Δ_R only existing, Eq. 3 will give:

$$R_2^I = -\frac{\Delta_R\,\omega}{\delta_R - \delta_L}\left[\frac{1}{1 + \frac{h\,\omega}{EA\,(\delta_R - \delta_L)}}\right] \ldots\ldots\ldots\ldots\ldots\ldots 6$$

It is thus disclosed that the general value of the reaction R_2^I is the algebraic sum of the two special reactions. Hence, if a certain loading produces a special reaction R_2^I and another loading the special reaction R_2'', each causing a corresponding set of stresses in each arm, the resultant reaction caused by the concurrent action of the two loadings will be the sum of the two special reactions and the stresses produced in the various truss members will be the algebraic sum of the stresses corresponding to those two special reactions.

APPLICATION TO BLACKWELL'S ISLAND BRIDGE.

All the preceding formulæ are perfectly general and applicable to any number of spans. In applying them to any special case, as that indicated by Fig. 3, due regard must be shown to the values and signs which Δ_L and Δ_R will then take. This may be illustrated by considering the particular structure shown by Fig. 3. There can be no shear or reaction R_2^I in the span l_2, Fig. 3, without a corresponding shear or reaction R_4^I in the span l_4; and the same observation is to be made regarding the deflections at those two points.

If d_2 is the deflection at C, Fig. 3, due to any arbitrary load, ω, (as 1 or 1000 pounds) acting at D in l_4, then the deflection at the same point C due to the shear or reaction R_4^I at D, expressed in the same unit ω, will be:

$$d_2 \frac{R_4^I}{\omega}.$$

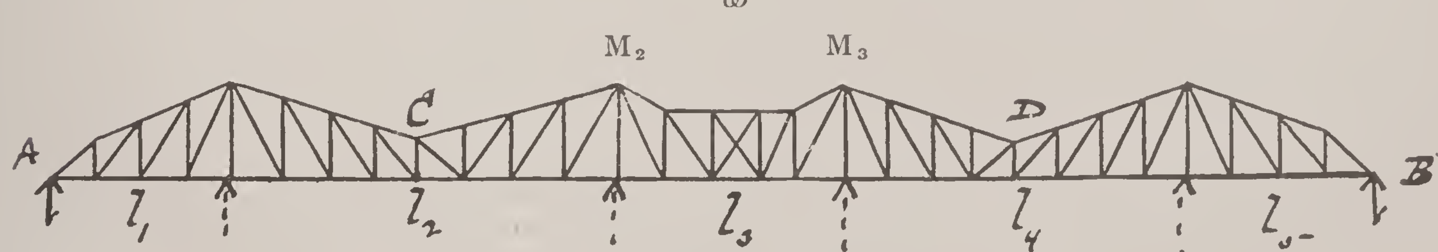


Fig. 3.

Hence in Eq. 3, there must be written for Δ_R the two terms $\Delta_R + \frac{d_2 R_4^I}{\omega}$; Δ_R now representing the deflection of the extremity of the

cantilever CM_2 due to any loads resting upon the structure between C and D. Similarly if d_4 represents the deflection of the extremity of the cantilever DM_3 due to the action of the same arbitrary load or unit at C, in the value of R^{I}_{4}:

$$R^{I}_{4} = \frac{(\Delta_R - \Delta_L)}{\delta_L - \delta_R}\left[\frac{\omega}{1 + \dfrac{h\,\omega}{EA\,(\delta_L - \delta_R)}}\right] \quad \ldots\ldots\ldots\ldots \quad 7$$

there must be written $\Delta_L + \dfrac{R^{I}_{2} d_4}{\omega}$ for Δ_L in Eq. 7; Δ_L now representing deflection due to load on CD.

There may then be written the following two equations by the aid of Eqs. 3 and 7.

$$R^{I}_{2} = \frac{\left(\Delta_L - \Delta_R - \dfrac{d_2 R^{I}_{4}}{\omega}\right)}{\delta_R - \delta_L}\left[\frac{\omega}{1 + \dfrac{h\,\omega}{EA\,(\delta_R - \delta_L)}}\right] \quad \ldots\ldots\ldots\ldots \quad 8$$

$$R^{I}_{4} = \frac{\left(\Delta_R - \Delta_L - \dfrac{R^{I}_{2} d_4}{\omega}\right)}{\delta_L - \delta_R}\left[\frac{\omega}{1 + \dfrac{h\,\omega}{EA\,(\delta_L - \delta_R)}}\right] \quad \ldots\ldots\ldots\ldots \quad 9$$

It must be carefully borne in mind that the same letter in the two equations 8 and 9 does not in general possess the same numerical value for any given structure, although in most cases *h* and *A* may have the same values.

Let there be placed in Eq. 8:

$$a_2 = \frac{\omega}{(\delta_R - \delta_L) + \dfrac{h\,\omega}{EA}}\ ; \quad \ldots\ldots\ldots\ldots \quad 10$$

Also, in Eq. 9:

$$a_4 = \frac{\omega}{(\delta_L - \delta_R) + \dfrac{h\,\omega}{EA}}\ ; \quad \ldots\ldots\ldots\ldots \quad 11$$

Then:

$$R^{I}_{2} = a_2\,(\Delta_L - \Delta_R)_2 - \frac{a_2 d_2 R^{I}_{4}}{\omega} \quad \ldots\ldots\ldots\ldots \quad 12$$

$$R^{I}_{4} = -a_4\,(\Delta_L - \Delta_R)_4 - \frac{a_4\, d_4\, R^{I}_{2}}{\omega} \quad \ldots\ldots\ldots\ldots \quad 13$$

Eliminating between Eqs. 12 and 13:

$$R_2^1 = a_2 \frac{(\triangle_L - \triangle_R)_2 + \frac{a_4 d_2}{\omega}(\triangle_L - \triangle_R)_4}{1 - a_2\, a_4 \frac{d_2\, d_4}{\omega^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 14$$

$$R_4^1 = - a_4 \frac{(\triangle_L - \triangle_R)_4 + \frac{a_2 d_4}{\omega}(\triangle_L - \triangle_R)_2}{1 - a_2\, a_4 \frac{d_2\, d_4}{\omega^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 15$$

The subscripts 2 and 4 affecting the parentheses in the numerators of the second members of Eqs. 14 and 15 indicate that the quantities $(\triangle_L - \triangle_R)$ belong to the spans l_2 and l_4 respectively.

Eqs. 14 and 15 give the two shears or reactions R_2^1 and R_4^1 at the extremities of the cantilever arms. When these two reactions are determined, the ordinary analytic or graphic methods for determining the stresses in all members of the entire structure from anchorage to anchorage may be applied. These operations were followed in computing the stresses shown on the sheets accompanying this report.

It will be observed that in Eqs. 14 and 15 the only variable quantities are $\triangle_R$ and $\triangle_L$ which are to be found by the aid of stresses determined by ordinary statical methods.

Wm. H. Burr,
Cons. Engr.

ALFRED P. BOLLER.
HENRY W. HODGE.

Telephone, 94 Rector.
Cable Address, "Bolodge."

BOLLER & HODGE,
CONSULTING ENGINEERS,
1 Nassau Street,
New York.

NEW YORK, October 28, 1908.

Hon. JAS. W. STEVENSON,
Commissioner of Bridges,
City of New York:

DEAR SIR—In accordance with the resolutions of the Board of Estimate and Apportionment, dated June 5th, and your instructions of June 9, 1908, we have made a careful investigation of the carrying capacity of the Blackwell's Island Bridge over the East River, and hand you herewith a full report of the results of this investigation, accompanied by ten drawings, and one appendix.

We beg to say that your Department has put all the records and data of this entire structure at our disposal, and has extended every assistance and courtesy in aiding us to arrive at the necessary facts on which our investigation is based.

Yours very truly,

BOLLER & HODGE,
Cons. Eng'rs.

H. W. H.

Boller & Hodge,
Consulting Engineers,
1 Nassau Street,
New York.

Report on the Carrying Capacity of the Blackwell's Island Bridge.

The Blackwell's Island Bridge in New York City, across the East River, extends from Sixtieth Street and Second Avenue, Borough of Manhattan, to Jackson Avenue and Jane Street, Borough of Queens, a distance of about 8,600 feet, made up of steel viaduct approaches at each end with a cantilever structure over the two channels and over Blackwell's Island, having a length of 3,724½ feet between anchorages, with spans as shown on Diagram No. 1.

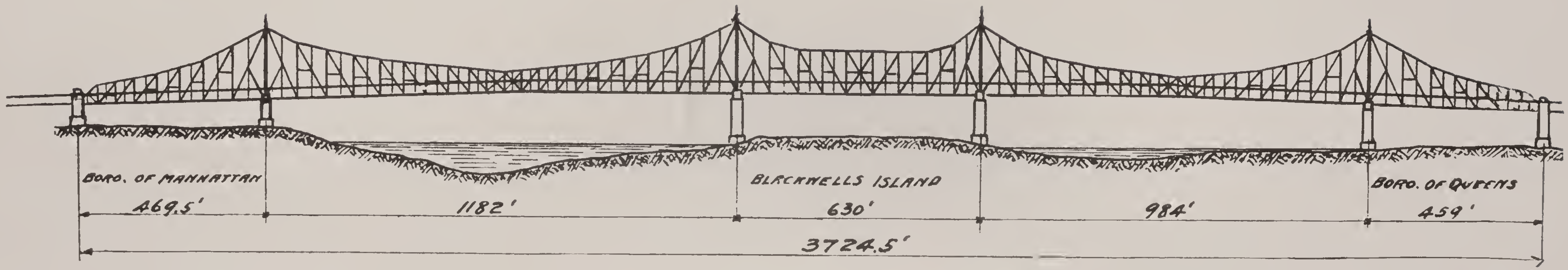

BORO. OF MANHATTAN
469.5'
1182'
BLACKWELLS ISLAND
630'
984'
BORO. OF QUEENS
459'
3724.5'

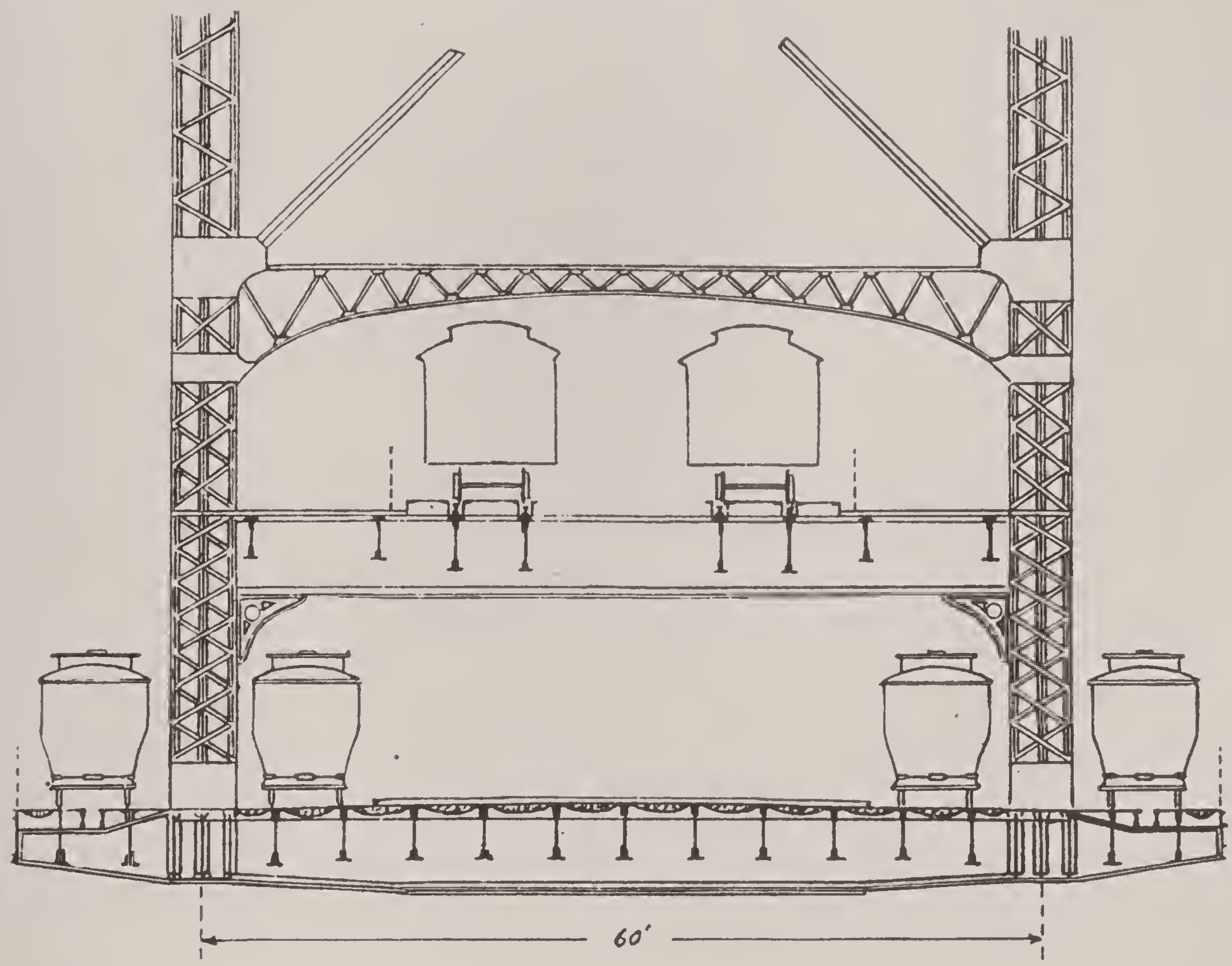
DIAGRAM NO. 1.

This cantilever structure differs from the usual type in having the lower ends of each shore lever arm attached by a rocker arm to the upper ends of each Island lever arm, thus making the entire structure continuous.

In accordance with our instructions we have confined our investigation to the cantilever structure, and all of our findings apply only to that portion of the bridge between the anchor piers.

This cantilever was originally designed to carry a 35½-ft. driveway, two 11-ft. sidewalks, four lines of trolley cars and two lines of elevated railway, arranged as shown on Diagram No. 2.

DIAGRAM NO. 2.



CROSS SECTION.

The contract for this structure was let to the Pennsylvania Steel Co. on November 20, 1903, and this contract design as shown by the lithographed album of drawings, together with the printed specifications issued with this album of drawings, will hereafter be referred to as the original design. It was estimated that the weight of the original design would be 84,300,000 lbs., made up as follows:

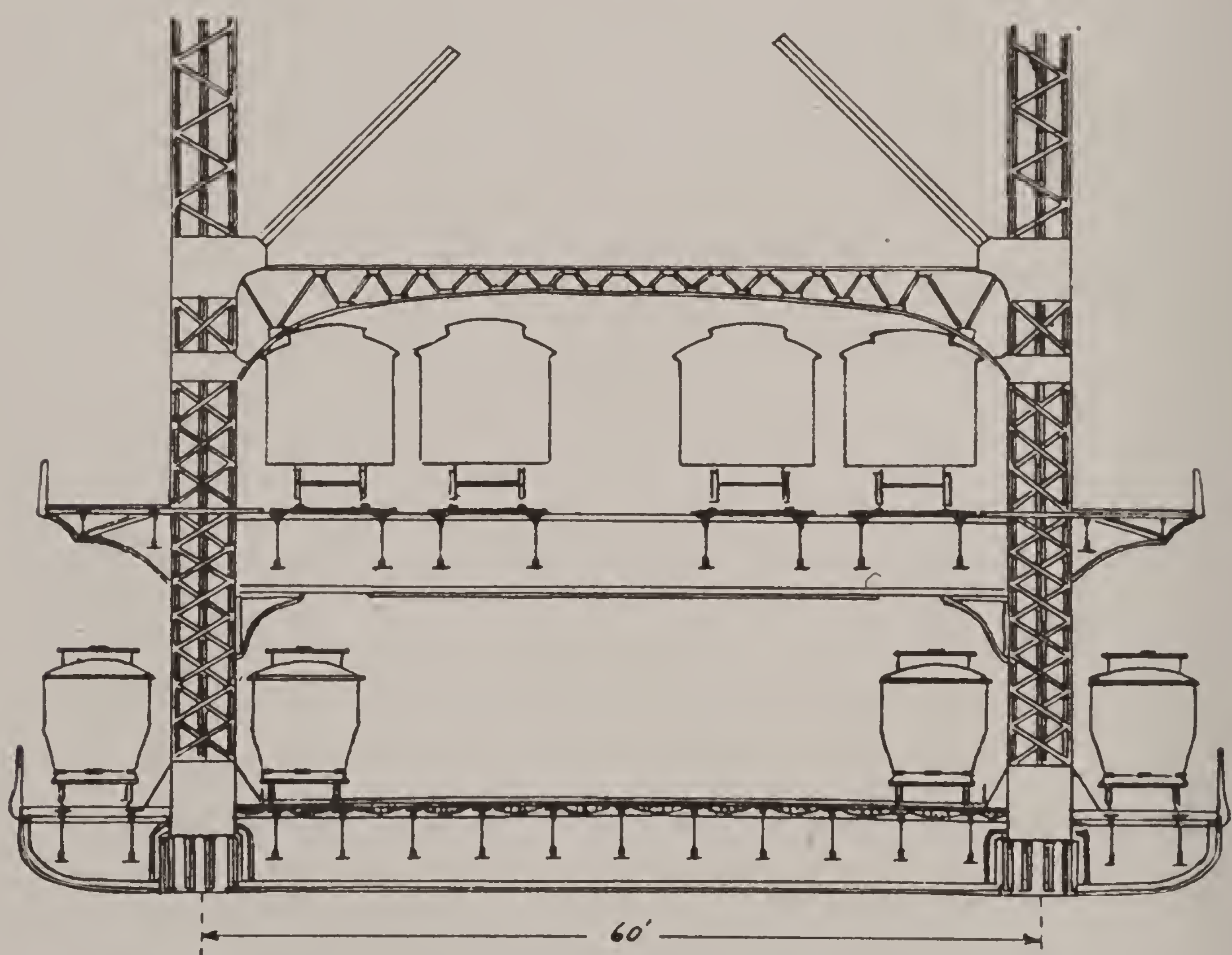
Nickel steel eye-bars	12,200,000 lbs.
Nickel steel pins	1,100,000 lbs.

Structural steel eye-bars	400,000 lbs.
Structural steel pins	50,000 lbs.
Structural steel, other than eye-bars and pins............	69,550,000 lbs.
Steel castings	1,000,000 lbs.
	84,300,000 lbs.

But the final sections and details had not been made when this estimate of weight was given, so the full data for an accurate estimate of weight did not then exist.

In September, 1904, it was decided to add two more lines of elevated railway to the original design, and these were to be arranged as shown on Diagram No. 3, the sidewalks being placed outside of the trusses.

DIAGRAM No. 3.

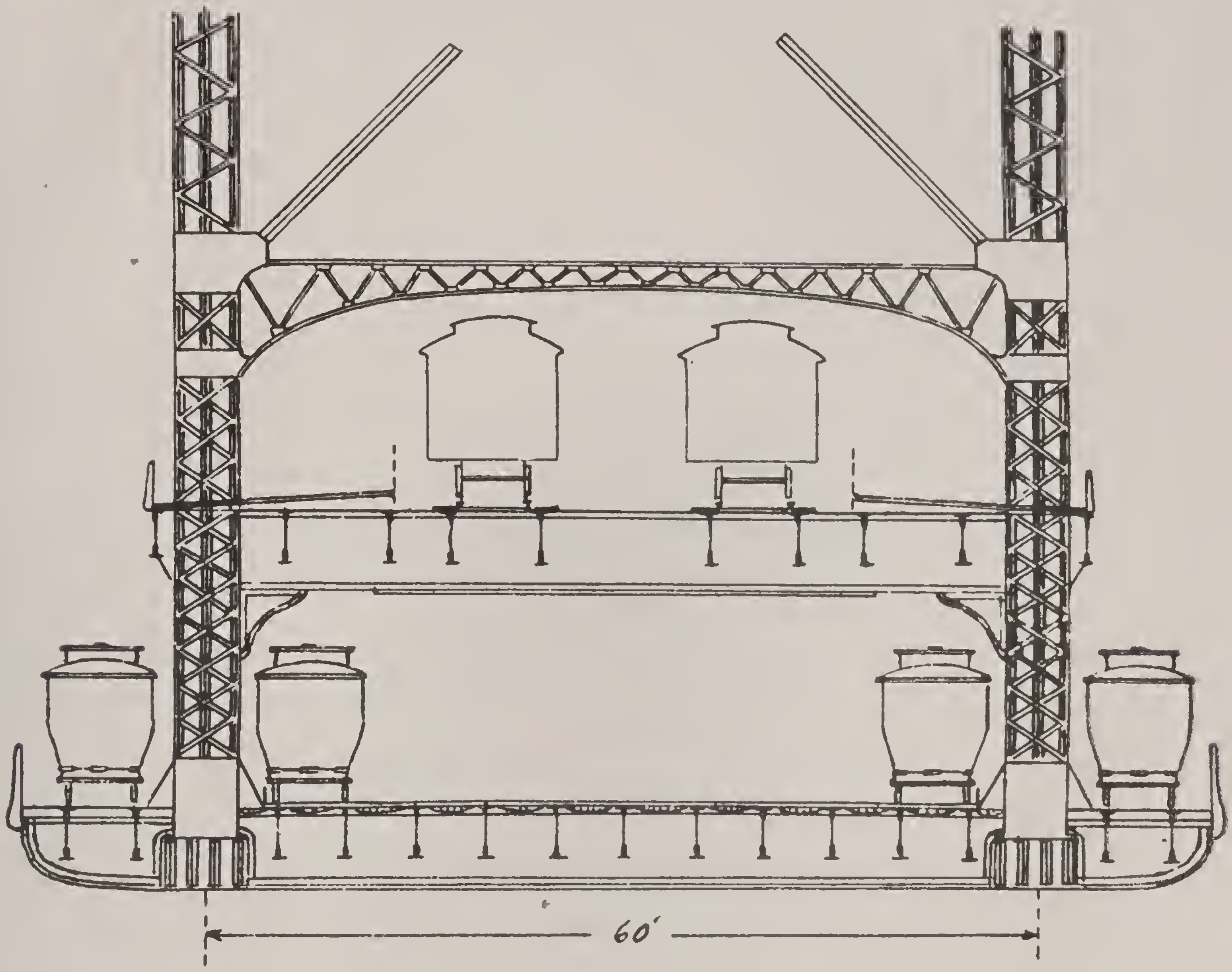


CROSS SECTION.

But it was determined that the four elevated tracks would not be immediately needed, so the bridge is now built with the outside footwalk stringers and the overhanging ends of the upper floor beams omitted, with

the sidewalks placed (for the present) in the place of the two outside elevated tracks, as shown on Diagram No. 4.

DIAGRAM NO. 4.



CROSS SECTION.

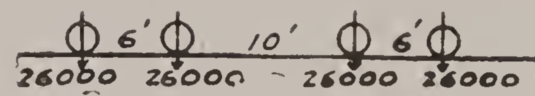
The original specifications called for a "congested" live load of 12,600 pounds p. l. f. bridge, made up as follows:

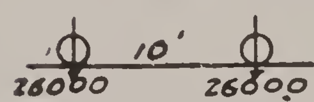
2 Elevated Railway Tracks at 1,700 lbs. p. l. f.	=	3,400 lbs. p. l. f.
4 Trolley Railway Tracks at 1,000 lbs. p. l. f.	=	4,000 lbs. p. l. f.
35½-ft. roadway at 100 lbs. per sq. ft.	=	3,550 lbs. p. l. f.
2 11-ft. sidewalks at 75 lbs. per sq. ft.	=	1,650 lbs. p. l. f.
		12,600 lbs. p. l. f.

and a "regular" live load of one-half the amount or 6,300 lbs. p. l. f.

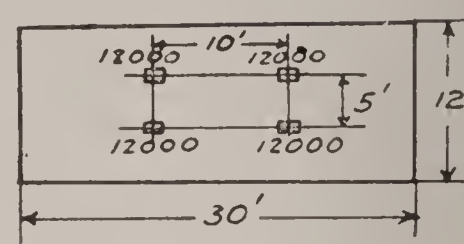
But when adding the two additional lines of elevated railway these loads were increased by the weight of these two additional tracks, giving a "congested" live load of 16,000 lbs. p. l. f. bridge, and a "regular" load of 8,000 lbs. p. l. f. bridge.

These uniform loads were specified for the main truss members only, and the floor systems and secondary truss members were to be designed for the following live loads:

On each elevated railroad track 

On each street car track 

or 1800 lbs. per lin. ft. of track.



On any part of the roadway 48000 lbs. on two axles 10′ apart and 5′ gauge covering a space 12′ x 30′, and 100 lbs. per sq. ft. on the remaining roadway surface.

On the footwalk a load of 100 lbs. per sq. ft.

The dead weight was specified to be the weight of the structure and floor, without any allowance for snow.

The wind load was specified to be 2,000 lbs. p. l. f. bridge, of which 1,000 lbs. was assumed to be a moving load and 1,000 lbs. a fixed load.

The specifications state that some of the eye bars and pins will be of nickel steel of the following chemical requirements:

	Per Cent., Max.
Phosphorus (Basic)	.04
Phosphorus (Acid)	.06
Sulphur	.05
	Per Cent., Min.
Nickel	3.25

The annealed specimens of this material were required to have the following physical values:

Elastic limit	48,000 lbs. minimum per sq. in.
Ultimate strength	85,000 lbs. minimum per sq. in.
Elongation in 8 inches	$\frac{1,600,000}{\text{ultimate.}}$

The full sized annealed bars of this material (up to a maximum size of 16 inches by 2½ inches) were required to show the following results:

Elastic limit	48,000 lbs. minimum per sq. in.
Ultimate strength	85,000 lbs. minimum per sq. in.
Elongation in 18 ft	9 per cent.

All other material in the structure was to be of open hearth steel, specimens (except rivets and steel castings) to show the following chemical results:

	Per Cent., Max.
Phosphorus (Basic)	.04
Phosphorus (Acid)	.08
Sulphur	.05

And to have the following physical values:

Elastic limit, plates and shapes	30,000 lbs. per sq. in. minimum.
Elastic limit, eye bars	½ of ultimate.
Ultimate strength, plates and shapes	60,000 lbs. desired.
Ultimate strength, eye bars	66,000 lbs. desired.
Elongation, per cent. in 8 inches	$\frac{1,500,000}{\text{ultimate strength.}}$

And annealed full-size eye bars to show results as follows:

Elastic limit	28,000 lbs. per sq. in. minimum.
Ultimate strength	56,000 lbs. per sq. in. minimum.
Elongation in body of bar	10 per cent.

We have examined the detailed reports of the mill inspectors on this material, and find that they show the metal fulfilled the above specifications.

With the above loads and quality of material, the following unit stresses were specified:

	For Dead Load and Regular Live Load or for Dead Load and Wind.	For Dead Load and Congested Live Load.
	Pounds Per Square Inch.	
For Nickel Steel in Eye Bars and Pins:		
Tension	30,000	39,000
Shear on pins....................	20,000	24,000
Bearing on diameter of pins........	40,000	48,000
Bending on outer fibre of pins......	40,000	48,000
For Structural Steel in Main Members of Trusses, Towers and Bracing:		
Tension	20,000	24,000
Compression	20,000-90 1/r*	24,000-100 1/r*
Shear on shop rivets, bolts and pins.	13,000	16,000
Bearing on diameter of shop rivets, bolts and pins..................	25,000	30,000
Bending on outer fibre of pins......	25,000	30,000

	Pounds Per Square Inch.
For Structural Steel in Secondary Members of Trusses:	
Tension in sub-verticals (hangers)..	18,000
Compression in sub-diagonals.......	18,000-80 1/r*
Shear on shop rivets and bolts......	12,000
Bearing on diameter of shop rivets and bolts	24,000
For Structural Steel in Floor System of Roadway and Footways and in all Floor Beams:	
Tension chords	15,000
Shear on shop rivets, bolts and web-plates, net section..............	10,000
Bearing on shop rivets and bolts....	20,000

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*Where 1=length and r=radius of gyration both in inches.

	For Dead Load and Regular Live Load or for Dead Load and Wind.	For Dead Load and Congested Live Load.
	Pounds Per Square Inch.	
For Structural Steel in Floor System (Including Brackets) for Railroad and Trolley Tracks:		
Tension chords	10,000	
Shear on shop rivets, bolts and web-plates, net section................	7,000	
Bearing on shop rivets and bolts....	14,000	
Allowable Pressure on Masonry:		
For dead load and regular live load..	550	
For dead load and congested live load	650	

These specifications and original contract drawings, which show the general dimensions of the bridge as built, form the basis of our investigation, and in addition thereto we used the detailed shipping invoices showing the exact scale weight of each and every piece in the structure, and the detailed shop drawings showing the areas of the sections.

We also computed, from the various contract drawings, the weight of the flooring material, including paving blocks, roadway concrete, sidewalk concrete, rails, ties and other materials for the various tracks, railings, pipes, wires and all other material required on the structure.

We also figured the amount of steel which will be required at some future time to complete the two sidewalks in their final position (as shown on Diagram 3) and all of these weights were found to be as follows:

	lbs. p. l. f. bridge.		lbs. p. l. f. bridge.
2 outside footwalk stringers	176		
Overhanging footwalk brackets	100	Add'l steel	403
Footwalk gratings	127		
2 upper outside railings	142		
2 upper inside railings	90		232
Reinforced concrete slabs for footwalk			500
Rails and contact rails for four upper tracks			330
Guard timbers and ties for four upper tracks			640
2 lower railings			174
Rails and conductor rails, 4 lower tracks			375
Wood paving blocks of roadway			1,041
Concrete under roadway paving blocks			3,200
Pipes, mail chutes, telephone, telegraph and feeder wires			405
			7,300

This total load of 7,300 lbs. p. l. f. bridge is referred to in this report and on the drawings as "additional material" being all the dead load except the shipped weight of structural steel.

From the above data we have carefully computed the live and dead stresses and unit stresses in each and every member of the structure, making our calculations on the following basis:

Dead Load.—We have taken for the dead load the scale weights of each piece as shipped and given on the various invoices, and we have apportioned these shipping weights to their proper panel points, arriving at the panel point dead loads as shown in detail on sheets 1 to 3, inclusive. To these scale weights of structural material, we have added the weight of "additional material" as heretofore given; this material being taken as uniform along the entire length of bridge, and apportioned to the panel points in proportion to the panel lengths. These two items constitute the entire dead load of the completed structure, and the result-

ing panel loads are shown in detail on sheets 1 to 3 and are summarized on sheets 4 to 6, which latter sheets show the points of application as used for the dead load calculations.

The actual shipping weight of steel now in the structure, as at present finished (as shown on Diagram 4) is 105,152,010 lbs., made up as follows:

Nickel steel eye bars	9,179,133 lbs.	
Nickel steel pins	1,460,563 lbs.	
Nickel steel links and pin plates	1,010,034 lbs.	
Nickel steel castings	119,900 lbs.	
Total nickel steel		11,769,630 lbs.
Structural steel eye bars	5,654,400 lbs.	
Structural steel pins	38,566 lbs.	
Structural steel other than eye bars and pins	84,795,779 lbs.	
Steel castings	2,253,094 lbs.	
Small iron castings	47,786 lbs.	
Cast-iron curb	592,755 lbs.	
Total structural steel		93,382,380 lbs.
Total weight		105,152,010 lbs.

This weight is distributed as follows:

Towers	12,633,200 lbs.
Anchorages	995,500 lbs.
Trusses, Bracing and Floor	91,493,310 lbs.
Total	105,152,010 lbs.

The dead load stresses have been figured on the assumption that both rocker arms will be adjusted when the entire dead load is in place, so that no dead load will pass through them, thus making the dead load stresses entirely independent of these rocker arms and making their values computable by the ordinary static methods.

Live Load.—In accordance with the terms of the specifications, we have assumed the live load to be "placed so as to give the greatest strain

in each part of the structure," and this condition requires that some sections of the bridge may be loaded and at same time other sections unloaded; for instance, the maximum compression in the rocker arms occurs with the two Island lever arms, and the two shore anchor arms loaded, and the other portions unloaded, and the maximum tension in these rocker arms occurs with the two shore lever arms and the Island span loaded and the other portions unloaded.

For the secondary members, except in a few cases where the bottom chord is not straight between adjacent main panel points, the live load stresses were figured with the local loads specified.

As the two rocker members connecting the ends of the lever arms cause the adjacent ends of the lever arms to move up and down together, the structure is continuous from end to end for live load stresses, and these stresses cannot be computed by the usual static method and must be found by means of the elastic properties of the materials.

While this method is well known and has been in use for some time, we give in an appendix an adaptation of it to this structure, which takes into account the simultaneous action of both the rocker arms and gives a simple and precise method of computing their stresses for any given loading.

This adaptation has been worked out by Mr. Clarence W. Hudson, who has had charge of these calculations for us.

With the stresses in the rocker arms known, the stresses in all the other members of the structure may be readily computed.

In computing the deflections of this structure, we have used the gross area of all riveted tension members. We have allowed nothing for play of pin holes, and we have not considered the tie plates, battens, or lattice on riveted members, nor the influence of the lateral system or the buckle plate floor. All of these matters have some slight influence on the actual deflections, but as it is only the ratio of certain deflections that is used to determine the stresses, to a certain extent, one influence offsets the other, and the stresses thus determined are not subject to serious error. We have computed deflections, using a modulus of elasticity of 28,000,000 lbs. for both carbon and nickel steel.

Wind Loads.—We have computed the wind stresses on the assumption that all wind pressure is transmitted by the transverse bracing

directly to the lower chord, the upper horizontal bracing being for vibration only.

We have computed the stresses for a fixed load of 1,000 lbs. p.l.f. over the entire structure and for an additional live load of 1,000 lbs. p.l.f. placed so as to give the greatest stress in each member.

The wind stresses have been computed by the same general method that was used for the live load stresses in the main trusses.

The formulæ for these wind stresses are similar in terms to those for the live load stresses, but differ in some of their signs due to the fact that a horizontal force at one end of either of the Island lever arms produces motion at the end of the other lever arm opposite in direction to the force.

We only show the chord stresses on our stress sheet as the web stresses cannot be given with accuracy since there is a solid buckle plate floor which carries a large portion of the wind shear.

Erection Stresses—We have computed the erection stresses using a traveller weighing 647 tons distributed as shown on sheet 10. This weight and spacing we have taken from the contractor's drawings (as this traveller had been removed before we started this investigation).

For the stresses in the Island span we have assumed a traveller on the East and West lever arm simultaneously, and we have taken traveller weights in such positions as to give maximum strains in every member, with the outer limiting position of the forward wheels 4 panels from the ends of the lever arms.

In addition to the stresses caused by the travellers, we have computed the simultaneous dead load stresses, assuming that all the weight of steel was in place, but not including the weight of "additional material" (amounting to 7,300 lbs. p.l.f. bridge), none of which was placed on the structure until after the removal of the travellers.

All erection stresses have been computed by simple static methods as the rocker arms were not rigidly connected till after the removal of the travellers.

Snow Load—The specifications do not call for any snow load on this structure, so that we have not figured any stresses for such a load, but in our opinion, a bridge of this character, with a practically solid lower floor 87 feet wide and an upper deck with two sidewalks and four lines of railroad track, should have been calculated for a considerable snow load.

We made a stress sheet for the loads called for in the specifications, but it was evident that the structure could not safely carry these loads, so we had to find the maximum safe carrying capacity of the structure. The final stresses under the conditions of safety hereinafter recommended are shown on our general stress sheet (Sheet No. 7) submitted herewith. This stress sheet also shows the effective area of each member, the final unit stresses, and the wind stresses in the chords.

The areas marked T for the main posts, U17-L17, U57-L57, U75-L75, and U107-L107, are the areas at their points of maximum width, and these posts decrease in area towards their ends, as the side plates keep the same thickness but decrease in width, giving the areas marked B at the bottom. The areas marked T for the other vertical posts are exclusive of the area of the transverse diaphragms in these posts, and the areas marked B are inclusive of these diaphragms.

We have made no additions for reverse stresses, as the specifications state that the sections are to be computed for the stress requiring the greatest area, so that the unit stresses here shown are the direct stresses from dead and live loads without any additions for reverse stresses, snow, wind, impact, or secondary stresses.

We have also computed the stresses in many of the floor beams and stringers for the upper and lower floors, using the local live loads specified, and for the cases computed we find the maximum flange stresses to be as follows:

Lower floor trolley stringers with cover plates	from 4,600 lbs. to 7,000 lbs. per sq. in.
Lower floor trolley stringers without cover plates.............	from 7,200 lbs. to 11,000 lbs. per sq. in.
Lower floor roadway stringers...	from 7,900 lbs. to 14,600 lbs. per sq. in.
Upper floor elevated railway stringers	from 6,500 lbs. to 9,100 lbs. per sq. in.
Upper floor sidewalk stringers...	from 4,100 lbs. to 10,800 lbs. per sq. in.
Upper floor floorbeams.........	from 13,000 lbs. to 14,400 lbs. per sq. in.
Lower floor floorbeams.........	from 15,000 lbs. to 16,000 lbs. per sq. in.

All of these stresses are for the static loads without impact.

The two pairs of outside trolley stringers on the lower floor are built of certain sections on the Island span and its lever arms, and of the same sec-

tions (for similar panel lengths), with additional cover plates on all other portions of the structure, thus making some portions of these tracks much stronger than others, as shown by the above stresses.

We are informed that the Island span and lever arm outside stringers had been completed for the specified trolley loads, when it was decided that subway trains might be run on these tracks and the remaining outside stringers were cover-plated. This, however, leaves these tracks without uniformity as to carrying capacity, unless cover plates are added to the outside stringers on the Island span and its lever arms.

The upper floor beams all have the same thickness of web, and the first cover runs the full length top and bottom, regardless of whether they carry a 20½-ft. panel or a 40-ft. panel, and this was undoubtedly done to carry the traveller during erection.

In a few of the lower floor beams and trolley stringers, the unit stresses exceed those specified; and this excess has been caused by their having been computed for a dead load less than the actual weight as finally called for by the flooring contracts, but this excess is so slight that, in our opinion, it will not affect their safety.

While the erection stresses are passed and can never recur again, we considered it advisable to compute these stresses to find if they had been greater than the stresses to which the structure may be subjected under traffic; and on sheet 10 we give the erection stresses for the chords and for such web members as will have the maximum erection stresses, though we did not consider it of sufficient value to compute all the minor web stresses from erection.

From this stress sheet we find that the maximum erection stresses in the chords do not equal the completed dead load stresses, and only in a few diagonals near the main piers do they equal the specified live and dead load stresses; so they furnish no data in the way of a full size test to determine the carrying capacity of the members, as the structure under traffic will be subjected to greater stresses than it was during erection.

We have examined the general design and details of the anchorages at the Manhattan and Queens ends and find the sections and the weight of masonry sufficient to carry the uplifts. We also find the pressure on the masonry of the main piers to be within safe limits.

We have not made a complete investigation of the secondary stresses in this structure, but we have made a limited investigation to approximately determine what the secondary stresses caused by temperature, distortion due to deflection, and bending due to own weight of members, amount to.

We find that the secondary stresses due to temperature are generally small, except in the post U75-L75, where in our opinion, the maximum stress from this source will occur, as this is the free end of the Island span, and a variation of $\pm 60°$ F. will move the top of this post and bend it around its fixed lower end, producing a fibre stress of 3,200 lbs. per sq. in.

The secondary stresses, due to distortion of the true figure of the trusses by the live load, are quite considerable, as the vertical deflection of the point L37 is 18 5/10 inches, and of the point L91, 14 2/10 inches for a live load of 3,000 lbs. p. l. f. truss.

We have made a careful analytical computation of the horizontal movement of the point U17 (and other similar points over the main piers) caused by this distortion, and for a live load of 3,000 lbs. p. l. f. truss in the position giving the maximum direct stress in the post U17-L17 we find this movement causes a fibre stress in the lower fixed end of U17-L17 of 2,600 lbs. per sq. inch, which is the maximum stress from this source, which occurs simultaneously with a maximum direct stress. The fibre stresses in the other similar posts are about the same, except that for U75-L75 it is reduced to 1,400 lbs. per sq. inch owing to the fact that this is the free end of the Island span; but this should be added to the temperature fibre stress in this post as above given. This distortion also causes some horizontal movement of the top ends of the rocker arms relative to their bottom ends and this relative movement causes additional secondary stresses, but the stresses found from this cause were so small as to be safely negligible.

The fibre stresses due to bending of members from their own weight depends to some extent on the total live load and dead load direct stresses on the members, but for a live load of 3,000 lbs. p. l. f. truss we find this fibre stress to be about 1,200 lbs. per sq. in. for many of the members, with extreme values running as high as 3,500 lbs. per sq. in.

There are also secondary stresses due to impact, and to bending in the vertical posts and hangers, caused by accelerating or retarding the moving loads on the upper floor, but we have not computed any values for these,

as we are of the opinion that they are negligible in this structure where the relative value of live load as compared with the dead load is so small.

The above maximum values of all these secondary stresses will probably not occur in the same members at the same time, but it will be seen that they may cause considerable increases in the direct unit stresses heretofore shown.

In addition to figuring the stresses on all members we have had a large number of the actual bridge members measured and calipered in the field, and we find that they agree with the sections we took from the shop drawings and used in these calculations, which sections we show in detail on sheets 8 and 9.

We have also computed the weight of a number of members from the shop drawings and find such weights agree with the shipping weights on the invoices, showing that the scale weights used for the dead load are correct.

We have figured the net sections of the riveted tension members from the shop detail drawings.

We have not carefully examined all the details of this structure, but we have checked the end connections of such members as are most heavily stressed and find them equal in strength to those members.

We have carefully considered the form and details of the lower chord as this feature has been criticised in the public journals, and the impression has been given that the lower chords in this structure are weaker than those of the Quebec Bridge, which, in our opinion, is not the case.

There is no full-size experimental data for the carrying capacity of such large compression members as are used in this structure, though the recent tests on models of compression members for the Quebec Bridge showed that such members when properly latticed carried 32,000 lbs. per sq. in. before failure, with a ratio of length to radius of gyration of 25. The only safety is to keep within the limits gained from experience on a smaller scale, as the means do not exist for learning the absolute carrying capacity of such sections, and a practical method of testing could not be provided in any reasonable time.

In our opinion, however, it is safe to follow the established practice for compression values, provided the limits set are not too high and the details are sufficient to make the member act as a unit and not fail in detail.

The heavier sections of the lower chord of this bridge are built up of four vertical webs, each 48 inches deep and varying in thickness with the sections required. Each web has an 8-inch by 6-inch angle, top and bottom, forming a "built-up channel." Each outside pair of these "built-up" channels is latticed together, top and bottom, with 5-inch by ⅝-inch double lacing bars, having two rivets in each end and one rivet at each intersection, the length of these lattice bars being about 45 inches, c. to c. of end rivets. This gives two separate built-up channel chords 48 inches deep and 25 inches b. to b. of angles. These two separate sections are then connected together with top and bottom tie plates 15 inches long and ½ inch thick, spaced about 5 feet, c. to c. The entire chord is thus about 70 inches wide, b. to b. of outside angles. While this chord is not as stiff transversely as it would be if properly latticed from outside rib to outside rib, and while in our opinion the lattice on the centre line, where the longitudinal shear due to bending is at its maximum, should not have been omitted, yet we believe that the chord is as strong horizontally as vertically for the following reasons:

The radius of gyration of the chord about its horizontal axis averages 14 inches and the radius of gyration of each outside latticed pair of channels around a vertical axis averages 12 inches, without any allowance for the connecting of these two pairs by the tie plates, which certainly add appreciably to the transverse stiffness, though the exact amount cannot be computed. However, the radius of gyration of the whole chord as a unit around a vertical axis is about 27, and while it would not be safe or proper to use this radius (owing to the omission of the central latticing), we consider it safe to assume that the tie plates add sufficient stiffness to increase the effective radius of gyration about a vertical axis from 12 to 14 and thus equal the vertical stiffness which of course sets the limit of stiffness for the section.

We have used for this chord a radius of gyration of 14, and all our results are based on this radius, and we are of the opinion that the details are sufficient to cause it to act as a unit with this radius.

The stress sheet shows that there are other compression members in the web system of the trusses which have higher unit stresses than the lower

chord, with a greater value of L/R, so the safety of the bridge is not alone determined by the value of this chord.

We have made a careful examination of the bridge as now completed, and find no evidence of loose rivets or buckling of members, or other indications of overloading, but there are four sub-diagonal posts C56-L57 on both trusses, and L107-C108 on both trusses, which had a " wind " in them during erection, and this was corrected by riveting a cover plate on the top of each post.

It will be noticed that both of these members are " sub-diagonals " which in no way affect the main stresses and are only for the support of one local panel load, and the stress sheet shows that they will never be subject to heavy stresses, so they are evidently safe and the " wind " was probably due to a bend in the shop or during erection, or to a slight over-run in length.

From the above data we come to the following

CONCLUSIONS:

First—That the specifications are clear and explicit and cover all the necessary requirements for the material and workmanship of a first-class structure, but in our opinion the working stresses given are in excess of good practice.

Second—That the steel manufactured for this structure is first-class bridge material and in accordance with the specifications.

Third—That the workmanship of this structure is first-class and in accordance with the requirements of the specifications.

Fourth—That the erection and field riveting of the structure appears to have been done in a first-class manner.

Fifth—That the actual sections of the various members agree with the sections ordered on the working drawings and shown on our sheets Nos. 8 and 9, and that the shipping weights are correct.

Sixth—That the members of floor systems are safe for the specified local live loads.

Seventh—That the following unit stresses in the main truss members, viz:

	Lbs. Per Square Inch.
Tension nickel steel bars	30,000
Tension structural steel bars and riveted members	20,000
Compression structural steel up to L/R = 20	19,000
Compression structural steel for L/R above 20	20,000-50 L/R

are the limit of safety for the direct stresses from the sum of the live and dead loads, as the secondary and snow load stresses heretofore referred to will add to these unit stresses and thereby increase the actual unit stresses.

Eighth—That the main truss members will not carry all the specified live loads for which this structure was designed.

Ninth—That the structure can safely carry a considerable live load in addition to its actual dead load.

As we, therefore, do not think the structure can carry all the specified live loads and yet is safe for considerable live load, we have tried to arrive at the actual live load which may safely come on this structure, using actual moving loads now in use on the subway, elevated, and surface lines of the city.

The heaviest trolley car now actually in use in this city has four axles with a loaded weight of 15,500 lbs. on each axle, and an over-all length of 42 ft. 6 in., giving an average load of 1,460 lbs. p.l.f. of each trolley track. The heaviest elevated or subway "motor" car has two loaded axles of 30,800 lbs. each, and two loaded axles of 22,200 lbs. each, and an over-all length of 51 ft., giving an average load of 2,080 lbs. p.l.f. track. The heaviest elevated or subway "trailer" car has four loaded axles of 17,500 lbs. each and an over-all length of 51.4 ft., giving an average load of 1,360 lbs. p.l.f. track.

The average weight of an elevated or subway train made up of five motor cars and three trailer cars is thus found to be 1,810 lbs. p.l.f. track.

The specified live load for the main trusses is one hundred pounds per sq. ft. on the roadways, and seventy-five pounds per sq. ft. on the foot-walks, and while we believe such loads are proper for designing local members of the floor system and secondary truss members, they are undoubtedly

excessive for the main truss members, as no such load can possibly come on the entire width of the roadway and sidewalks for any considerable length of the structure. For the main truss members, we, therefore, consider a live load of fifty pounds per square foot on the roadway and sidewalks to be fully sufficient.

By trial, we find that the main trusses will safely carry a live load of three thousand pounds per lineal foot of each truss, if the dead load be reduced by one thousand pounds per lineal foot of each truss. Some of this dead load, consisting of the outer sidewalk stringers, floor-beam brackets, sidewalk gratings, and track material for the railway tracks on the upper deck has never been put on the structure, and some of the dead load now on, can be omitted, thus decreasing the full dead load and permitting the use of the above live load. Furthermore, the average live load on the trolley tracks can be reduced by regulating the minimum spaces between same, as these units are so small that for the main trusses the load is practically uniform for the cars spaced at reasonable intervals. For the elevated railway tracks it is not possible to greatly reduce the effective load per lineal foot by regulating the intervals between trains, as these trains are long and heavy units, and if trains are spaced at intervals of three or more train lengths, they will fall in such positions as to practically produce the maximum stresses in important parts of the structure.

With these live loads, the roadway and footwalks produce a live load of 17¾ ft.+11 ft.=28¾ ft.@50 lbs.=1,437 lbs. per lineal foot of truss, and if we assume each trolley track to be loaded with cars at clear intervals of one car length or about 85 ft. c. to c., they will produce a live load of 2×730 lbs.=1,460 lbs. per lineal foot of truss, making a total live load of 2,897 lbs. per lineal foot of truss, which would require a reduction in the dead load of about 1,000 lbs. per lineal foot of truss to meet the conditions of safety, and this reduction can be readily made.

We, therefore, come to the conclusion that it is safe to run the four lines of trolley cars on this bridge as at present constructed, if the portions of the dead load not yet in place, and not required for highway or trolley service, be omitted, and if the four lines of inside track stringers on the upper deck be removed.

The "additional" dead load then will consist of the following items:

	Lbs. p. l. f. bridge.
Concrete sub-pavement of roadway......................	3,200
Wood block sub-pavement of roadway..................	1,040
Rails & conductor rails for 4 trolley tracks...............	374
Concrete footwalk slabs	800
6 railings	406
Electric wires, gas mains, etc.	160
	5,980
Less deduction for removing 4 lines of stringers..........	680
	5,300

or 2,650 lbs. p. l. f. each truss, which is a reduction of 1,000 lbs. p. l. f. each truss from the full assumed dead load.

It will be noted that the concrete footwalk slabs are here taken at 800 lbs. p. l. f., whereas they were taken at 500 lbs. p. l. f. in the full dead load; this is due to the fact that in the completed structure they are intended to be 10 ½′ wide, whereas those now temporarily placed on the outer lines of railway stringers are 16′ wide. To reach the full reduction required, it will also be noted that we have reduced the allowance for wires and pipes to 160 lbs. p. l. f. bridge.

With a live load of 50 lbs. per square foot on the roadway and footwalks, and a live load of 780 lbs. per lineal foot of each trolley track (which is equivalent to running trolley cars at intervals of 80 feet c. to c.) we get a total live load of 3,000 lbs. per lineal foot of each truss, and with this live load in addition to the total assumed dead load reduced by 1,000 lbs. per lineal foot of truss, we have made the complete stress sheet herewith submitted (Sheet No. 7) and it will be seen that all these stresses come practically within the limits of safety set by us, and the bridge is in our opinion safe for these loads.

It will be noted that the dead load stresses shown on this stress sheet are computed with the full weight of structural material shown on our sheets 1, 2 and 3, but the total dead panel loads shown on our sheets

1 to 6 inclusive were reduced for computing these dead load stresses by a uniform deduction of 1,000 lbs. p. l. f. truss, as above itemized.

The dead load of the floor systems can be further reduced if necessary, and we are of the opinion that by reducing the dead loads on all the lever arms, and perhaps increasing the dead loads on the Island span and anchor arms, the structure can be made to safely carry one pair of elevated tracks.

In our opinion the most practical method of getting this bridge into prompt service is to remove the stringers from the two inside lines of upper railway tracks, and open the bridge for service of the highway, sidewalks and four trolley tracks immediately.

If at some future time it is found necessary to make use of one pair of elevated tracks, further modifications of the dead loads can be made and the outer lines of elevated railway stringers used for their true purpose; the sidewalks being removed to their final position outside the trusses.

We therefore make the following

RECOMMENDATIONS:

First—That the stringers of the two inside tracks on the upper deck, or other equivalent dead load, be removed to lighten the dead load.

Second—That the trolley traffic be so regulated that if four tracks are in use the cars shall not run with clear intervals between them of less than their own length.

Third—That the bridge be opened for the traffic of the sidewalks, highway and four lines of trolley tracks as at present constructed, subject to the above recommendations.

Fourth—That if any other moving loads be added to the structure, such further modifications of the dead load shall first be made as will keep the total direct stresses caused by the live and dead loads within the safe limits herein set.

Under these recommendations we are confident the structure is perfectly safe.

Respectfully submitted,

BOLLER & HODGE,
Consulting Engineers.

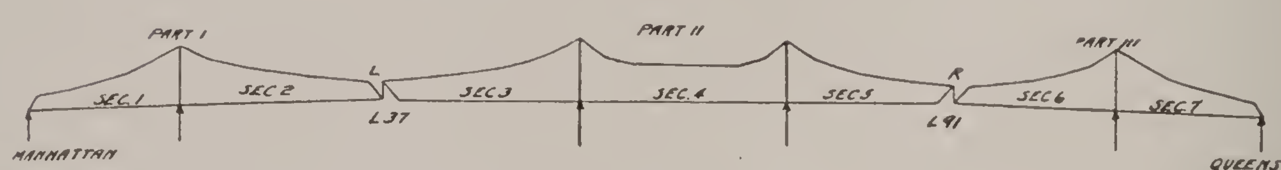
October 28, 1908.

Accompanied by one appendix and ten drawings, as follows:

1. Detailed dead panel loads, Manhattan Anchor and Lever Arm.
2. Detailed dead panel loads, Island Span and Lever Arms.
3. Detailed dead panel loads, Queens Anchor and Lever Arm.
4. Dead panel loads and points of application, Manhattan Anchor and Lever Arms.
5. Dead panel loads and points of application, Island Span and Lever Arms.
6. Dead panel loads and points of application, Queens Anchor and Lever Arms.
7. General stress sheet of main trusses.
8. Sections of truss members, Manhattan and Queens Anchor and Lever Arms.
9. Sections of truss members, Island Span and Lever Arms.
10. Erection stress sheet.

APPENDIX.

Formulae for Determining the Stresses in the Rocker Arms of the Blackwells Island Bridge.



Let PL = stress in the left rocker arm.

Let PR = stress in the right rocker arm.

Let d1 = deflection of point L37 of the Manhattan lever arm, due to a load of unity at L37, the rocker arm being considered disconnected at the bottom end.

Let d2 = deflection of point L37 (the bottom of the left rocker arm) of the Blackwell's Island west lever arm, due to a load of unity at L37, the rocker arms being considered disconnected at their bottom ends.

Let d3 = deflection of point L91 (the bottom of the right rocker arm) of the Blackwell's Island east lever arm, due to a load of unity at L91, the rocker arms being considered disconnected at their bottom ends.

Let d4 = deflection of point L91 of the Queens lever arm due to a load of unity at L91, the rocker arm being considered disconnected at the bottom end.

Let d20=deflection of point L 37 of the Blackwell's Island West lever arm due to a load of unity at L 91, both rocker arms being disconnected at their bottom ends.

Let d30=deflection of point L 91 of the Blackwell's Island East lever arm due to a load of unity at L 37, both rocker arms being disconnected at their bottom ends.

Let D1=deflection of point L 37 of the Manhattan lever arm, due to the given loading, the bottom end of the rocker arm being considered disconnected.

Let D2=deflection of point L 37 of the Blackwell's Island West lever arm, due to the given loading, both rocker arms being considered disconnected at their bottom ends.

Let D3=deflection of point L 91 of the Blackwell's Island East lever arm, due to the given loading, both rocker arms being considered disconnected at their bottom ends.

Let D4=deflection of point L 91 of the Queens lever arm, due to the given loading, the bottom end of the rocker arm being considered disconnected.

The above deflections it will be noted are all for bottom chord points.

The deflections for the Manhattan and Queens anchor and lever arms should always include that due to the anchor bars.

The deflections for the Blackwell's Island anchor and lever arms should include that due to the rocker arms.

The rocker arms L and R if disconnected at their bottom ends divide the structure into three portions, which are each statically determinate. These three portions we will designate as Part I, Part II and Part III.

Part I. we will divide into Section No. 1, Manhattan Anchor Arm, and Section No. 2, Manhattan Lever Arm.

Part II. we will divide into Section No. 3, Blackwell's Island West Lever Arm, and Section No. 4, Blackwell's Island Span, and Section No. 5, Blackwell's Island East Lever Arm.

Part III. we will divide into Section No. 6, Queens Lever Arm, and Section No. 7, Queens Anchor Arm.

Parts I. and II. When the rocker arm L is connected move up and down together and the bottom end of L, which is a common point for both parts, has the same motion.

Parts II. and III. When the rocker arm R is connected move up and down together, and the bottom end of R which is a common point for both parts has the same motion.

It is also true that with a live load on Part I. and L and R connected, the members of Part I. can only receive stress from the load and from the rocker arm stress PL produced by this load, the members of Part II. can only receive stress from the stresses PL and PR and the members of Part III. can only receive stress from the stress PR.

For a live load on Part II.:

The stresses in Part I. are due to PL.

The stresses in Part II. are due to the load and PL. and PR.

The stresses in Part III. are due to PR.

For a live load on Part III.:

The stresses in Part I. are due to PL.

The stresses in Part II. are due to PL. and PR.

The stresses in Part III. are due to PR. and the load.

A careful analytical determination of the several deflections shows that:

A vertical downward force of unity will make d1, d2, d3, d4, d20 and d30, all downward.

A vertical downward live load in Section No. 1 will make D1, upward.

A vertical downward live load in Section No. 2 will make D1, downward.

A vertical downward live load in Section No. 3 will make D2, downward, and D3, downward.

A vertical downward live load in Section No. 4 will make D2, upward, and D3, upward.

A vertical downward live load in Section No. 5 will make D2, downward, and D3, downward.

A vertical downward live load in Section No. 6 will make D4, downward.

A vertical downward live load in Section No. 7 will make D4, upward.

We may now write equations for the stresses PL and PR in terms of known quantities (PL and PR will be taken as in tension or plus until their sign is known).

Take first a load on Section No. 1. We know that the motion of the bottom ends of both L and R is upward.

The upward motion of L (bottom end) for section No. 2 is $= D1+PLd1$.

The upward motion of L (bottom end) for section No. 3 is $= -PLd2-PRd20$.

These must be equal $\therefore D1+PLd1 = -PLd2 - PRd20$ or $PL(d1+d2) = -D1-PRd20$.

The upward motion of R (bott. end) for section No. 5 is $= -PRd3-PLd30$.

The upward motion of R (bott. end) for section No. 6 is $= PRd4$.

These must be equal $\therefore -PRd3-PLd30 = PRd4$ or $PR(d3+d4) = -PLd30$.

Take now a load on section No. 2. We know that the motion of the bottom ends of both L & R is downward.

The downward motion of L (bott. end) for section No. $2 = D1-PLd1$.

The downward motion of L (bott. end) for section No. $3 = PLd2+PRd20$.

These must be equal $\therefore D1-PLd1=PLd2+PRd20$ or $PL(d1+d2) = D1-PRd20$.

The downward motion of R (bott. end) for section No. $5=PRd3+PLd30$.

The downward motion of R (bott. end) for section No. $6 = -PRd4$.

These must be equal $\therefore PRd3 + PLd30 = -PRd4$ or $PR(d3 + d4) = -PLd30$.

Take now a load on Section No. 3. We know that the motion of the bottom ends of L and R is downward.

The downward motion of L (bott. end) for section No. $2 = -PLd1$.

The downward motion of L (bott. end) for section No. $3 = D2 + PLd2 + PRd20$.

These must be equal $\therefore -PLd1 = D2 + PLd2 + PRd20$ or $PL(d1 + d2) = -D2 - PRd20$.

The downward motion of R (bott. end) for section No. $5 = D3 + PRd3 + PLd30$.

The downward motion of R (bott. end) for section No. 6 = — PRd4.

These must be equal ∴ $D3 + PRd3 + PLd30 = - PRd4$ or $PR(d3 + d4) = - D3 - PLd30$.

Take now a load on section No. 4. We know that the motion of the bottom ends of L and R is upward.

The upward motion of L (bottom end) for section No. 2=PLd1.

The upward motion of L (bottom end) for section No. 3=D2—PLd2—PRd20.

These must be equal ∴ $PLd1 = D2 - PLd2 - PRd20$ or $PL\ (d1+d2) = D2 - PRd20$.

The upward motion of R (bottom end) for section No. 5=D3—PRd3—PLd30.

The upward motion of R (bottom end) for section No. 6=PRd4.

These must be equal ∴ $D3 - PRd3 - PLd30 = PRd4$ or $PR\ (d3+d4) = D3 - PLd30$.

For a load on each of the three remaining sections, we may write two simultaneous equations for finding PL and PR.

For a load on section No. 5, we have:

$$PL\ (d1+d2) = -D2 - PRd20.$$
$$PR\ (d3+d4) = -D3 - PLd30.$$

For a load on section No. 6, we have:

$$PL\ (d1+d2) = -\ PRd20.$$
$$PR\ (d3+d4) = D4 - PLd30.$$

For a load on section No. 7, we have:

$$PL\ (d1+d2) = -\ PRd20.$$
$$PR\ (d3+d4) = -\ D4 - PLd30.$$

The above seven pairs of equations may be very much simplified by solving them for PL and PR.

The solved equations are:

For a load on section No. 1, $PL = -\ M\ D1.$
$PR = +N\ D1.$

For a load on section No. 2, $PL = +M\ D1.$
$PR = -\ N\ D1.$

For a load on Section No. 3, $PL = -\ M\ D2 + N\ D3.$
$PR = -\ G\ D3 + N\ D2.$

For a load on Section No. 4, $PL = +M\,D2 - N\,D3$.

$PR = +G\,D3 - N\,D2$.

For a load on Section No. 5, $PL = -M\,D2 + N\,D3$.

$PR = -G\,D3 + N\,D2$.

For a load on Section No. 6, $PL = -N\,D4$.

$PR = +G\,D4$.

For a load on Section No. 7, $PL = +N\,D4$.

$PR = -G\,D4$.

In which $M = \dfrac{d3+d4}{(d1+d2)\ (d3+d4) - d20d30}$

$$G = \frac{d1+d2}{(d1+d2)\ (d3+d4) - d20d30}$$

$$N = \frac{d20}{(d1+d2)\ (d3+d4) - d20d30} = \frac{d30}{(d1+d2)\ (d3+d4) - d20d30}$$

The numerical values of the deflections should be used in the above formulae without regard to their signs.

The resultant signs for the solved equations will give the character of stress in the rocker arms; + indicating tension, and — compression.

It will be noted that the values M. G. and N. are constants for the structure, so that they can be computed once for all and used for any and every position of the moving load.

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